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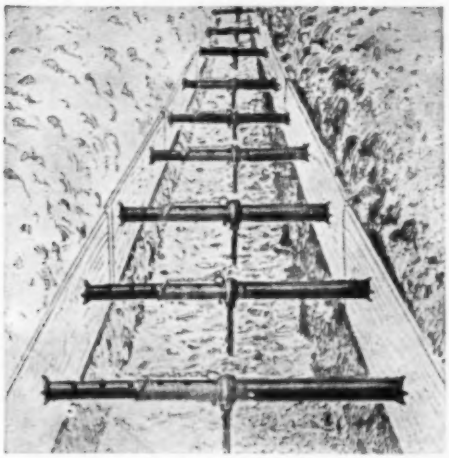

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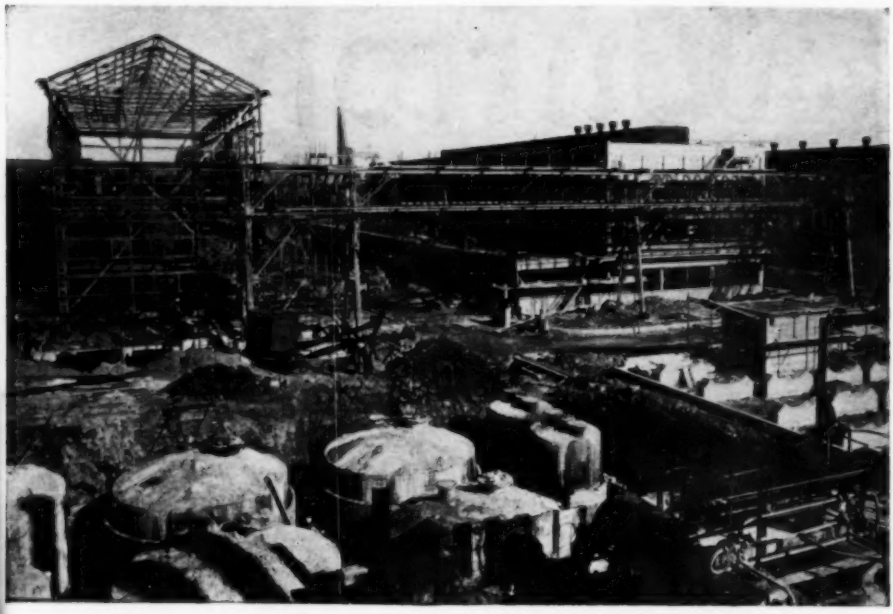

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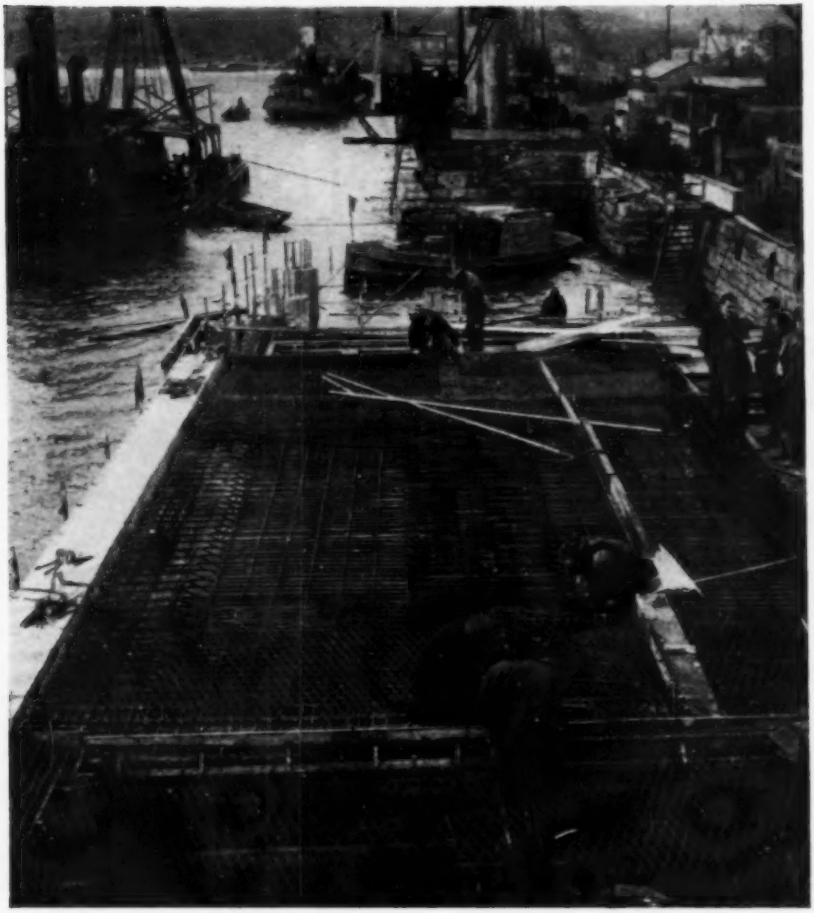

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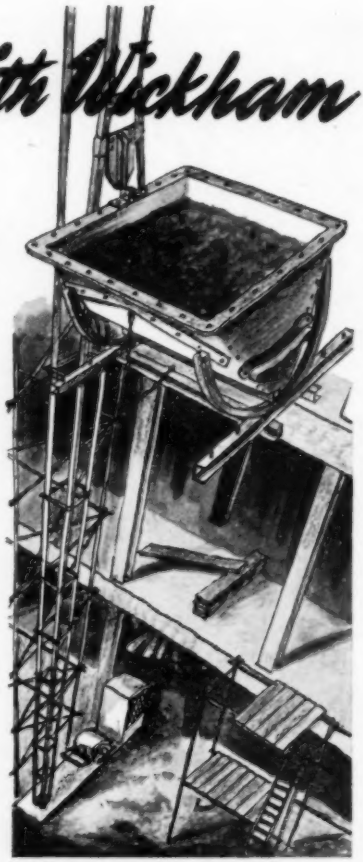

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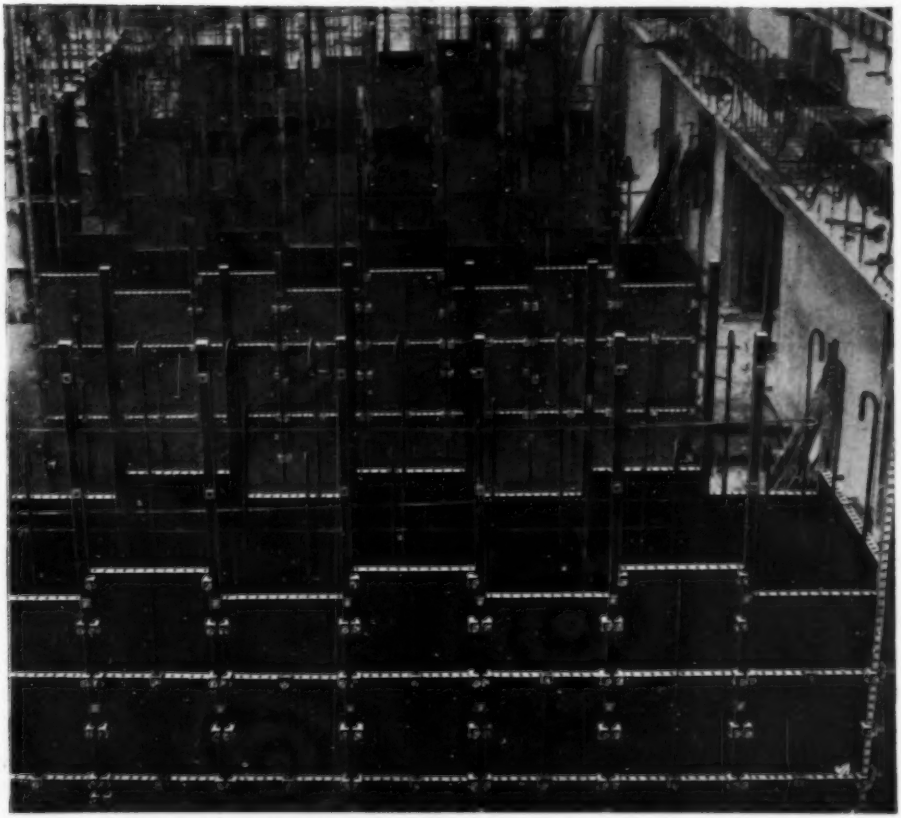

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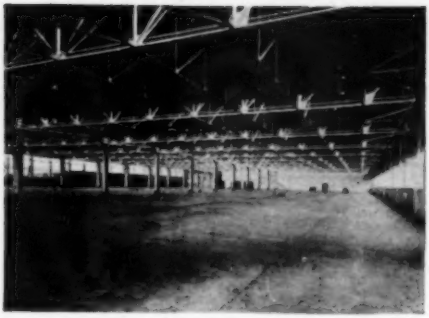

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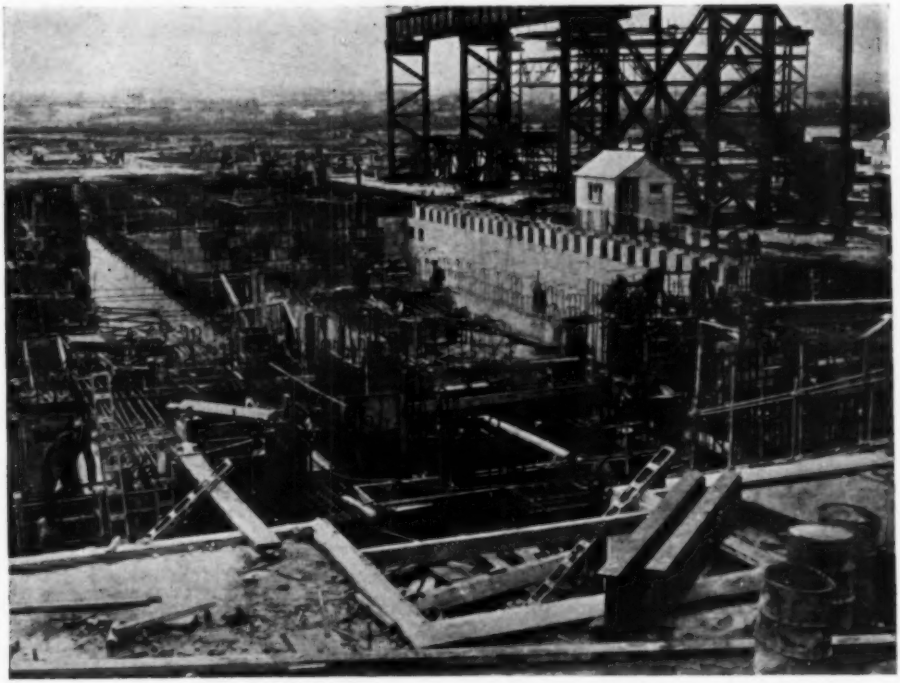

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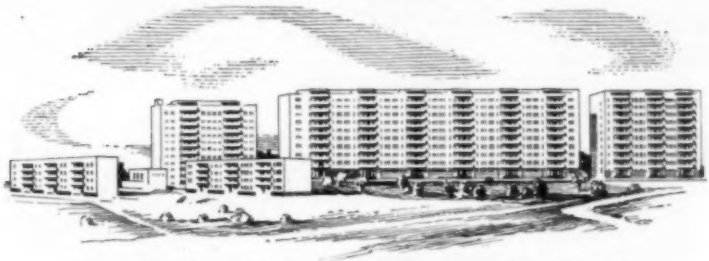

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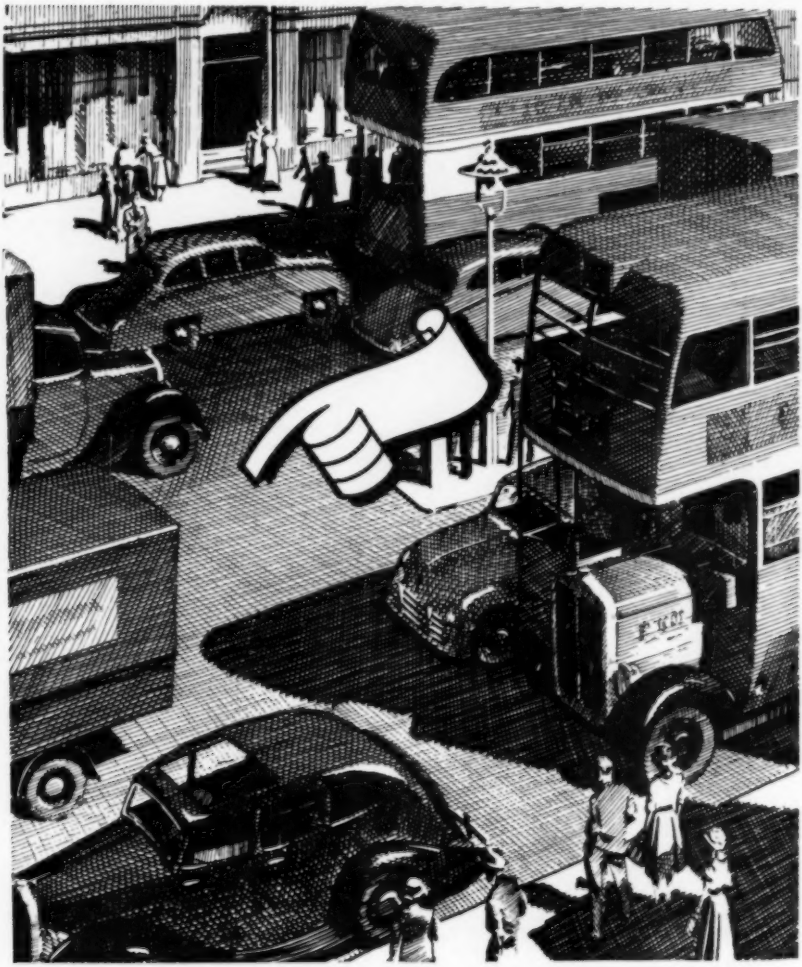

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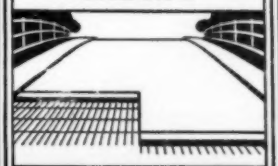

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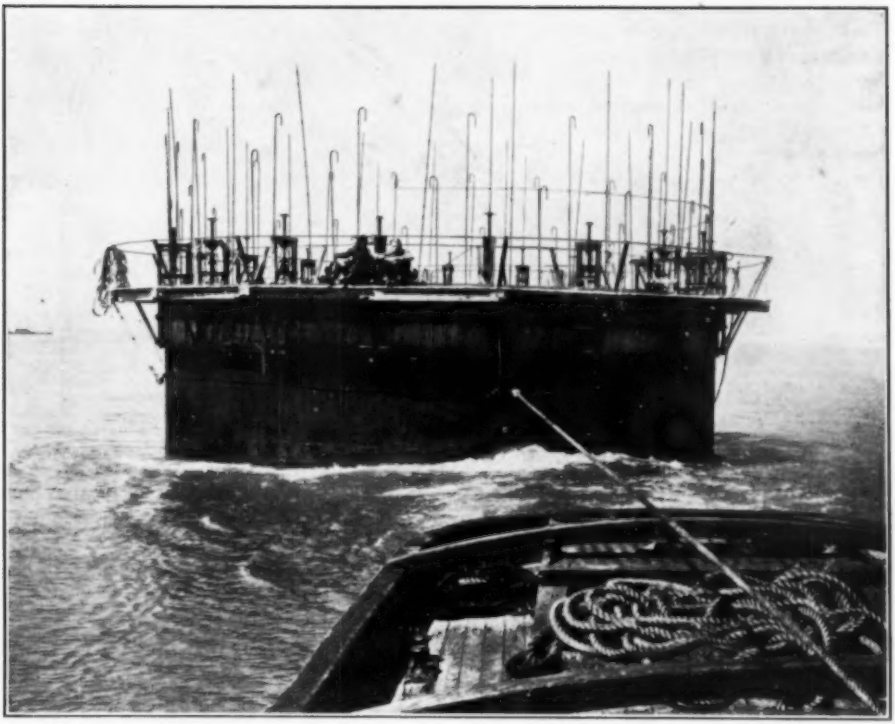

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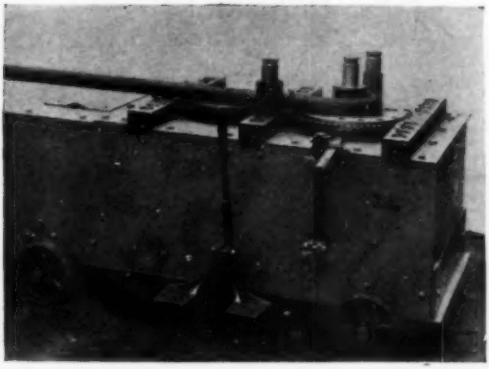

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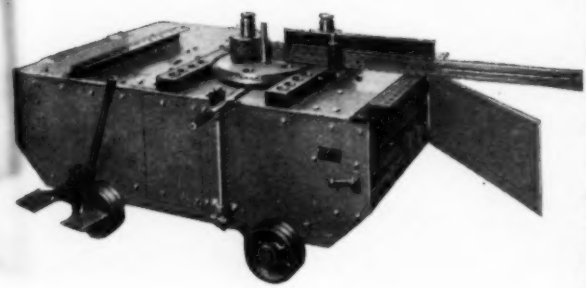

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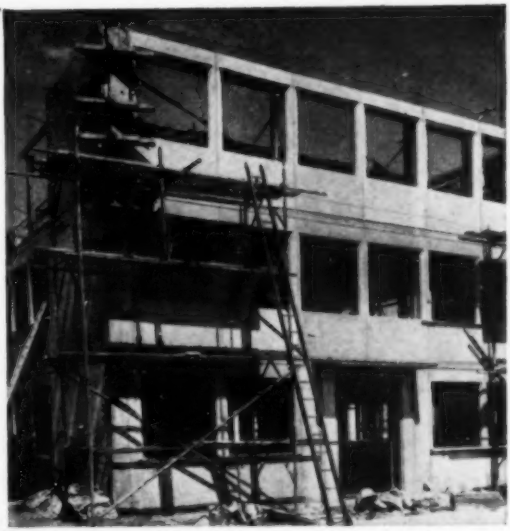

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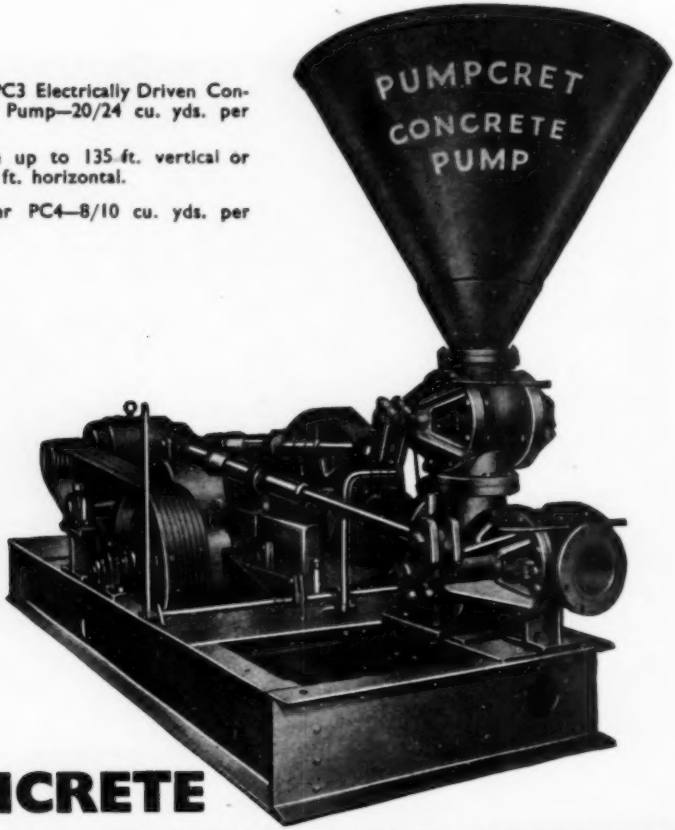

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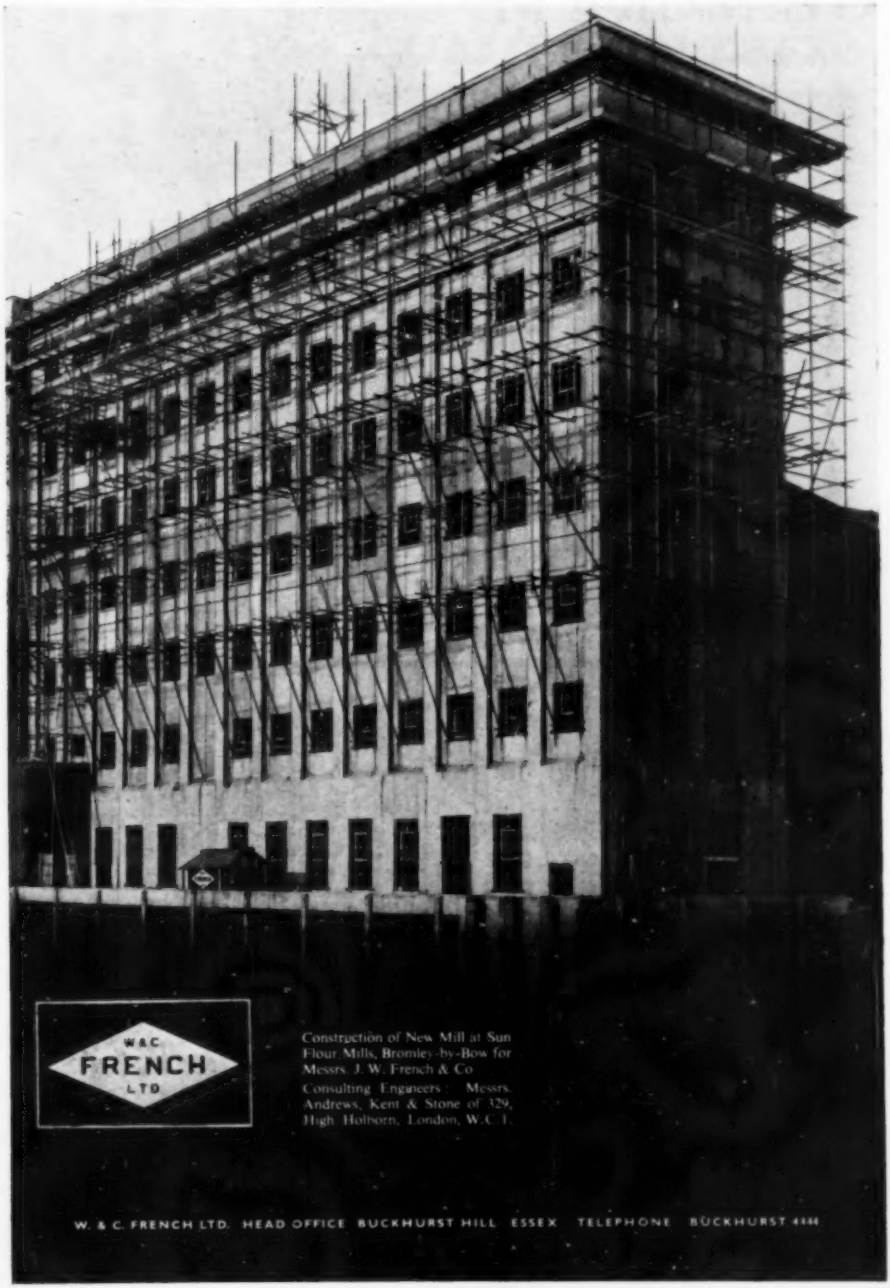
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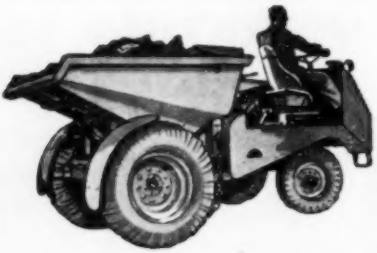

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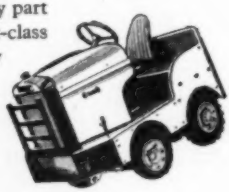

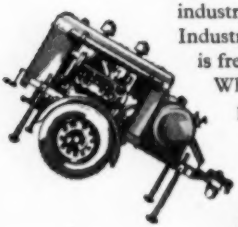

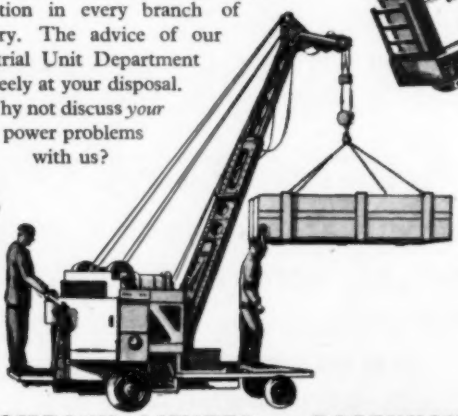

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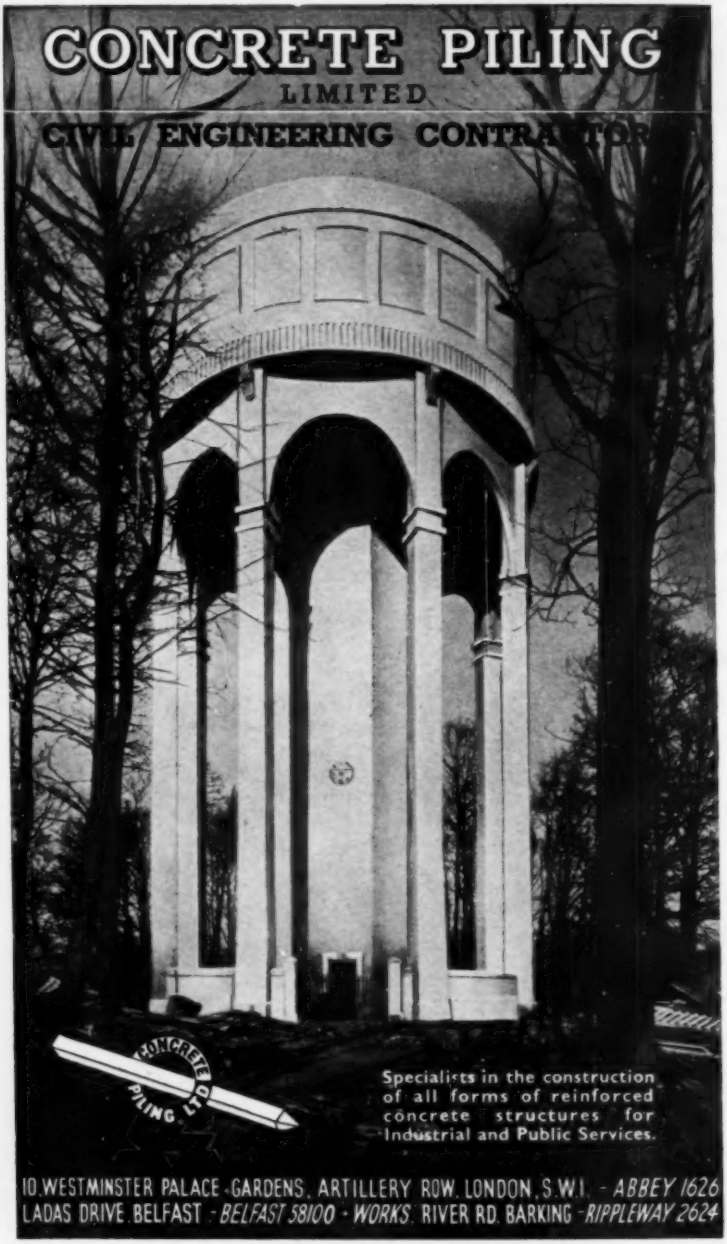
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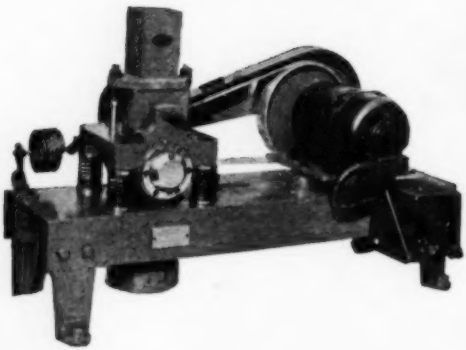

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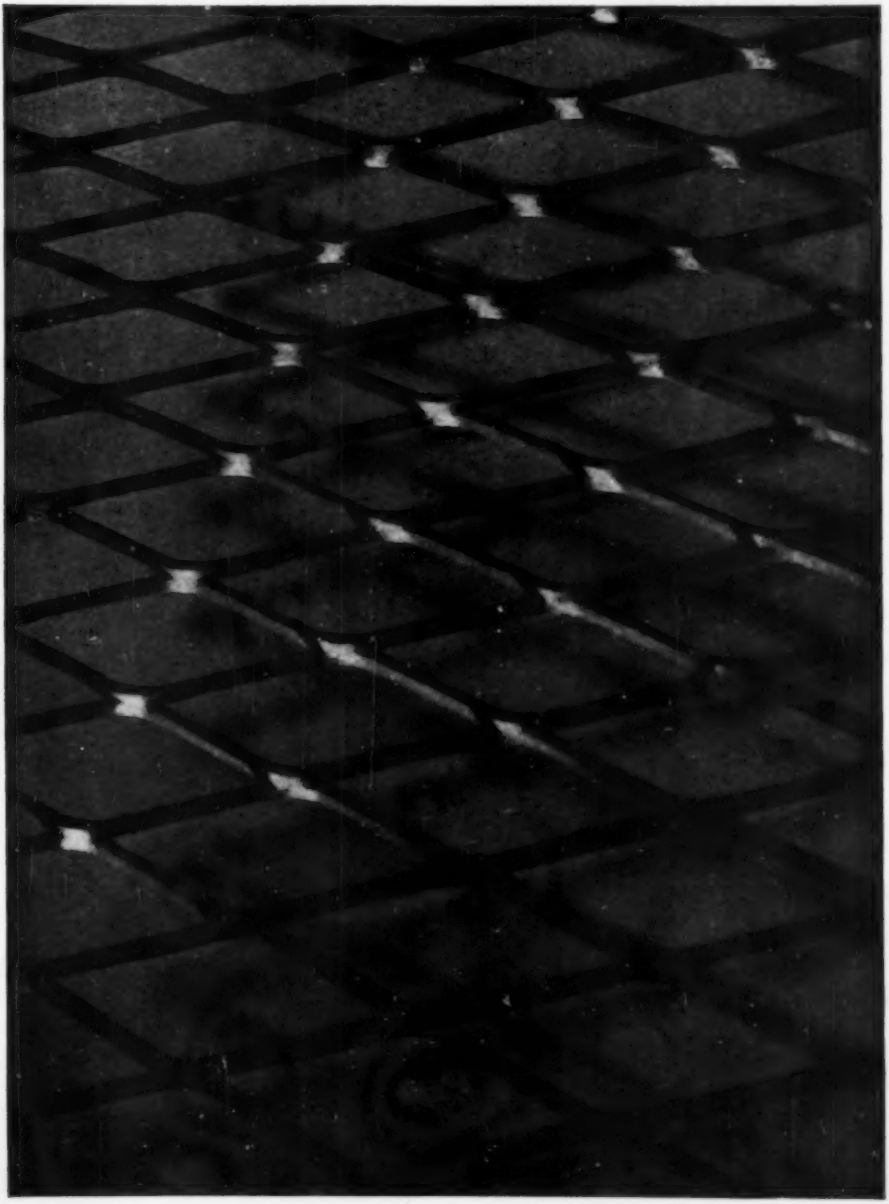

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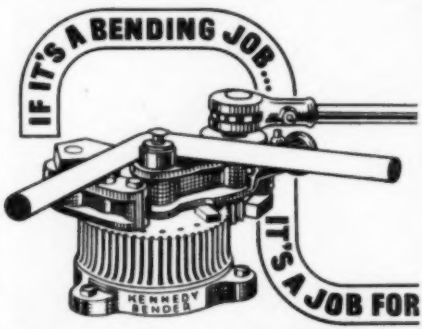

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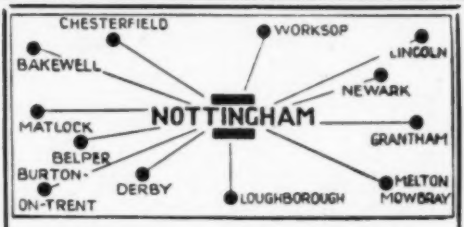

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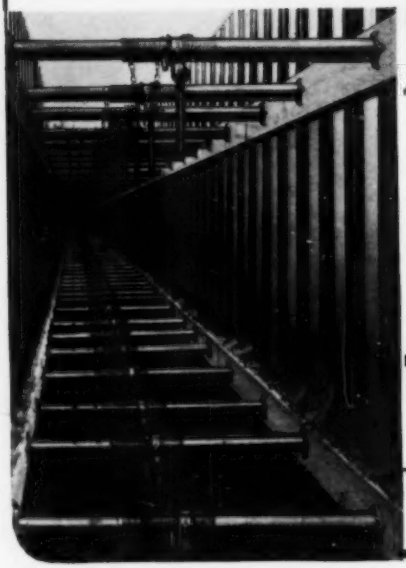

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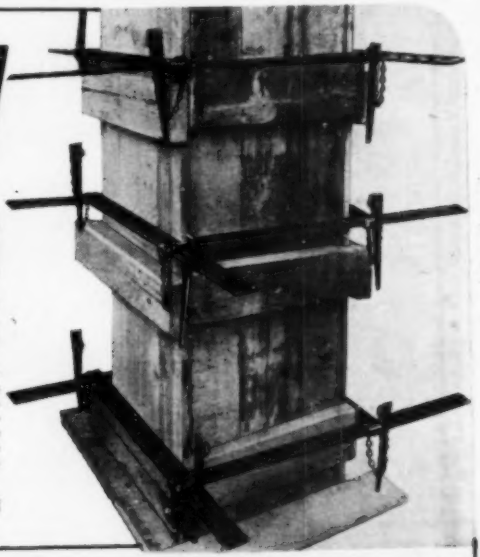

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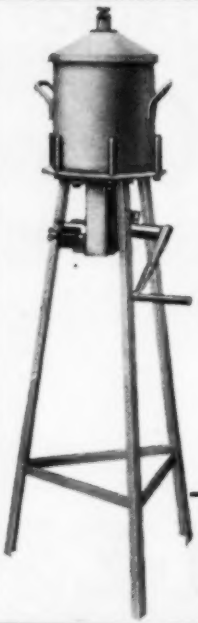

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Volume XLVII. No. 3. LONDON, MARCH, 1952.

EDITORIAL NOTES

Plain Words.

IN his book " ABC of Plain Words ", which has been written to guide Civil servants in their choice of words when writing letters, Sir Ernest Gowers uses the expression " ABC lay-out " for alphabetical arrangement, he advises his readers to say that a restriction is " lifted " when it is ended, and in his preface he writes, " Over a large part of the field that should be plain sailing " in a context which might lead some people to write " Generally this will be easy ", and so use fewer words but at the expense of not amusing the reader. Much can therefore be excused in documents prepared by engineers and which do not pretend to be guides to the choice of words. We have in the past occasionally pointed out examples of carelessly-drafted documents prepared by committees of professional people. Sometimes the fault seems to be due to an attempt to use legal phraseology, with all its pitfalls. Sometimes it is due to using words in common use (or misuse) in one profession without realising that a word to which an unusual meaning is given for a special purpose means to others only what a dictionary says it means. In a few cases the cause is ignorance of the meaning of words. Clear writing indicates clear thinking. At a time when some engineers are anxious that members of their profession should have more influence in the affairs of the nation, it is desirable that they should have a reputation for logical and clear exposition as well as for logical and clear thinking. It is for this reason that we venture to make the following comments on the " First Report on Prestressed Concrete " issued recently.

This report is one of the best-written documents of its kind that we have seen for some years, yet in such an excellent work as this some words and expressions might with advantage be changed. For example, the word job is not a happy description of a structure or a structural member. The statement that " The concrete strength at transfer should be at least 2½ times the maximum compressive stress " seems to mean that the concrete should be 2½ times as strong as its actual strength ; it is probably intended to say that the compressive strength should be 2½ times the greatest stress assumed in the calculations. It is stated that the " reaction is taken up by the hardened concrete ", whereas the concrete provides the resistance but takes up nothing. The expression " pre-tension stresses " in steel can only, by the ordinary meaning of words, refer to stresses before the steel is tensioned, whereas the term means the stresses in the steel after it has been tensioned. This confusion is no doubt due to the use of the term

prestressed concrete to describe compressed concrete ; the prefix is unnecessary, and has come into use only because the term " précontraint " used in France has been followed without thought elsewhere. The " minimum possible " effective prestress must be no prestress ; the least probable prestress appears to be intended. There seems to be confusion in the use of the words anchor and anchorage. The anchor in prestressed concrete is the device, such as a cone or wedge, which fixes the ends of the wires. The anchorage is the specially-strengthened end of, say, a beam, or a steel plate, or a buttress in which the anchors are secured. The expression " failure of anchorage " is probably intended to include the failure of the anchors, which might fail independently of the anchorage. The use of nouns as adjectives is best avoided ; the moment of resistance of the concrete is better than " the concrete moment of resistance ", and crushing of the concrete is better than " concrete crushing ". Why is concrete adjacent to steel called " fibres " ?—it is undesirable that any of the substances defined by a dictionary as fibres should be present in prestressed concrete. Deduction is not always a good substitute for calculation. To deduce means to infer, or to come to a conclusion, as a result of reasoning, which is a different process than is used when, in the words of the report, we " deduce from direct measurement ".

Also it would be useful to give reasons for some of the recommendations, or perhaps to add a few words of explanation. For example, the use of calcium chloride to hasten the hardening of concrete is mentioned as though it is common practice, but in view of test results that show that there is a risk of this chemical causing corrosion of steel it is doubtful if all engineers would permit its use in concrete in contact with steel wires of small diameter. Words such as appropriate, appreciable, and due can mean such different things to different people that, as the report is published for guidance, some indication might be given of what is meant by, for example, " an appreciable variation in length ". Why should each size of aggregate be kept in separate bins on the site if properly-graded material can be obtained without this extra expense ?

The word " transfer " to describe the operation of applying the compressive force to the concrete is hardly a suitable one. Transfer implies movement, and in this operation nothing is moved except that the device that keeps the steel taut is transferred from the anchorage to a bench. What is the operation or the time that it is desired to indicate ? A wire has been stretched and is held in tension. If we " transfer " this force from the wire to the concrete, then we have tension in the concrete and no tension in the wire, because we have transferred it from the one to the other. What happens in fact is that the tension (or most of it) remains in the wire, and the concrete is compressed. It can, of course, be said that the reaction to the tensile force in the steel is transferred from, say an external abutment to the concrete by bond or through the anchorage, but this meaning is not clearly implied by the use of the word transfer to describe the time when the concrete is compressed.

It may be said that these few suggestions about some of the words and expressions used in such an excellent report are of no importance because technical men will know what is meant. But even so some time and thought are necessary before it is realised, for example, that " pre " means after, and there is no reason why anyone should have to think twice about the meaning of plain words.

The Design of Prestressed Concrete Beams for Working Conditions.

By P. W. ABELES, D.Sc.(Vienna), M.I.Struct.E.

THE distribution of stress in a prestressed concrete beam at working load is entirely different from the distribution near failure. The ultimate resistance was considered by the writer in this journal for October, 1951. Under ordinary conditions of loading the distribution is rectilinear as in an elastic homogeneous material, and this distribution can be assumed even if fine cracks have developed when temporary nominal tensile stresses may be considered. This assumption is justified because, after cracking, prestressed concrete beams show complete resilience, and the ultimate resistance is not diminished by repeated loading. The cracks close, however, when the load is reduced to the value at which theoretical tensile stresses disappear, at which stage the ordinary straight-line distribution is resumed ; a different distribution of stress applies so long as the cracks are open. In this journal for April, 1948, the writer gave formulæ for the design of beams at working load and these are extended in this article.

Notation.

The notation is in accordance with the " First Report on Prestressed Concrete " issued by the Institution of Structural Engineers, with some additional symbols. Generally the suffixes (1) and (2) relate respectively to the bottom and top of the beam, which are the planes (or " fibres ") of a simply-supported beam at which the maximum tensile and compressive stresses respectively occur. In continuous beams and poles, in which tensile or compressive stresses may occur at either face, one face should be selected as plane (1) and the opposite as plane (2). If a few sections only are to be investigated, it is preferable to assume that (1) relates to the tensile face of each section. If (1) is not the tensile face, the formulæ must be adjusted.

Two cases are considered, namely, at the time the compressive force is applied to the concrete (this stage is generally called " at transfer "), and at working load. The suffix t indicates either the resultant stresses at transfer or the stresses while the beam is being transported, because during transport conditions may be less favourable and must be investigated separately. It is usual to prestress a beam when it is the right way up. During tensioning of the wires the middle part of the beam rises off its bed until it is supported at the ends only, and stresses contrary to those produced by the prestressing force develop due to the weight of the beam. At transfer (and also during lifting and transporting) the resultant reduced stresses only need be considered if the beam is always kept the right way up and supported only at the ends, but this applies mainly to beams with post-tensioned cables because the cables can be bent up and a suitable distribution of stress can be obtained at the ends where the stresses due to the weight do not counteract those due to the prestressing force. This arrangement is more difficult with beams with pre-tensioned wires and it is preferable to design such beams, and any beams that are cast upside down, so that they can be lifted in any manner.

The notation additional to that in *Fig.* 1 is as follows.

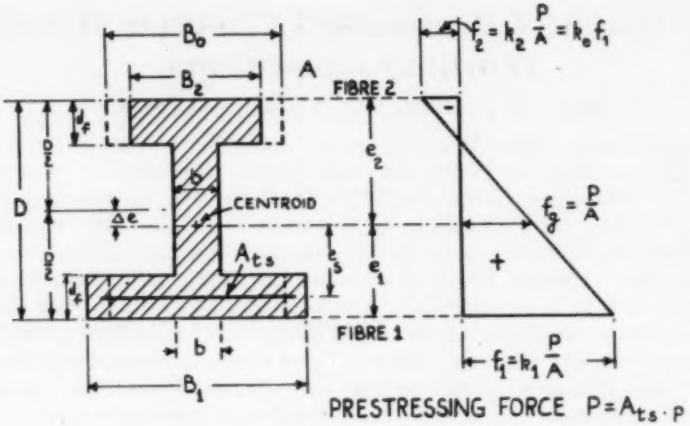


Fig. 1.

$A_t (= A_{ts} + A_{tu})$, the cross-sectional area of the steel in the bottom of the beam which may be composed of tensioned wires or bars (A_{ts}) and untensioned reinforcement (A_{tu}).

$d_{av.}$, the average depth of A_t below the top of the beam.

f_{1T}, f_{2T}, the stresses in the concrete at transfer at faces (1) and (2) respectively due to prestressing alone.

f_{1t}, f_{2t}, the resultant stresses in the concrete at faces (1) and (2) respectively at transfer or during transport, for example the stress f_{1T} (or f_{2T}) plus the stress due to the weight of the beam.

f_{1e}, f_{2e}, the effective stresses in the concrete at faces (1) and (2) respectively due to prestressing alone, after the maximum reduction of the force in the wires or bars has occurred.

f_{1w}, f_{2w}, the resultant stresses in the concrete at faces (1) and (2) respectively under working load, based on f_{1e} and f_{2e} respectively. (Note.—Compressive stresses are positive; tensile stresses are negative.)

$f_{2t(lim.)}$, the limiting resultant stress at face (2) at transfer if no tensioned wires are provided near this face.

f_{ct}, f_{tt}, the permissible compressive and tensile stresses respectively in the concrete at transfer.

f_{cw}, f_{tw}, the permissible compressive and tensile stresses respectively in the concrete at working load. (Note.—Permissible stresses are positive.)

f_{1m}, f_{2m}, the permissible moment stresses at faces (1) and (2) respectively, which depend on the permissible stresses in the concrete at transfer and at working load and on $f_{2t(lim.)}$.

f_g, the stress in the concrete at the centroidal (or gravity) axis.

$M_w (= M_s + M_a = M_d + M_l)$, the bending moment due to the total working load, that is the sum of the bending moments due to self-weight of the beam and additional load, or the sum of the bending moments due to the dead and live loads.

P, P_i, P_t, P_e, the general, initial, transfer, and effective prestressing forces in the steel in the bottom of the beam; the corresponding stresses are p_i, p_t, and p_e; the similar forces in the top of the beam are P', P'_i, P'_t, and P'_e.

R_t, the ratio of the prestress at transfer to the initial prestress.

R_e, the ratio of the prestress effective after the maximum possible reduction of the initial prestress has occurred to the initial prestress.

$R_o = \left(\frac{p_e}{p_t} = \frac{R_e}{R}\right)$, the ratio of effective prestress to the prestress at transfer.

$r\left(= \sqrt{\frac{I}{A}}\right)$, the radius of gyration.

$Z_1\left(= \frac{I}{e_1}\right)$, $Z_2\left(= \frac{I}{e_2}\right)$, the section moduli related to the bottom and top faces respectively.

$Z_0\left(= \frac{B_0 D^2}{6}\right)$, the section modulus of a rectangular section of width B_0.

Permissible Stresses in the Concrete.

The permissible compressive stress f_{ct} in the concrete at transfer should not exceed 30 per cent. to 40 per cent. of the crushing strength of cubes of the same age in order to avoid excessive creep. The compressive stress f_{cw} at working load should, according to the Report, not exceed one-third of the cube strength, which seems to be a satisfactory limit but may not ensure that the ultimate compressive resistance, which must be investigated separately, is sufficient. The permissible tensile stress f_{tt}' at transfer must generally be such that cracks are avoided near the anchors. To ensure a factor of safety against cracking, f_{tt} should, according to the Report, not exceed one-half the modulus of rupture near the anchors. With regard to the permissible tensile stress f_{tw} at working load, three types of beams have to be distinguished, namely (i) beams in which the possibility of cracking at working load must be avoided, (ii) beams in which temporary cracks are permissible under the greatest live load if this load rarely occurs, and (iii) beams in which visible hair cracks are permissible under live loads that occur frequently. Beams of type (i) may be subdivided into beams (iA) in which tensile stresses should be avoided (for example beams subjected to impact), and beams (iB) in which tensile stresses less than the modulus of rupture are permitted. According to the Report, f_{tw} in beams subject to fatigue should not exceed one-half the modulus of rupture, which is a very safe recommendation since, according to tests made at the Eidgenössische Material Prüfungs Anstalt at Zurich, the ratio of the cracking load in dynamic tests to that in static tests is 85 per cent. to 90 per cent.[1] A tensile stress of 70 per cent. to 75 per cent. of the modulus of rupture should ensure freedom from cracks under constant load, and a stress of 90 per cent. of the modulus of rupture may still ensure freedom from cracks under occasional loads, as mentioned in the Report.

Objections to beams of type (iB) are sometimes made because tensile stresses are permitted in the concrete, the bases of the objection being that cracks due to shrinkage or settlement may have occurred before the application of the prestress and that cracks may be caused by accidental loads in excess of the working load. It may be said, however, that cracks due to shrinkage should not occur in beams with pre-tensioned wires since the length of the beam is limited by practical considerations. If post-tensioning is applied to beams cast in situ or to precast beams of extraordinary length, thin separation joints, representing artificial cracks, can be provided at sections where the bending moment is considerably less than the maximum and where tensile stresses do not occur under working load. With

(1) "Prestressed Concrete," by F. G. Thomas (page 33). Conference on Prestressed Concrete held at the Institution of Civil Engineers, February, 1949.

regard to cracks due to excessive loads, the results of fatigue tests[1] on partially-prestressed composite beams should be considered. In these tests the beams were first statically loaded until several hair cracks appeared. After one million repetitions of loading corresponding to stresses of 100 lb. per square inch in compression and exceeding 600 lb. per square inch nominally in tension, the cracks closed entirely on removal of the load. In another test three million repetitions of loading were applied, at each million the loading being increased. In both cases the ultimate resistance under static load was not reduced in spite of the previous loadings. These severe tests show that if hair cracks occur and open under subsequent fatigue loading, they remain hair cracks and close completely when the load is removed.

The behaviour of a prestressed concrete beam under working load depends on the magnitude of the prestressing force. If the force is very great, cracking and failure may occur almost simultaneously. With a smaller force the factor of safety against cracking is reduced but the resilience is greatly increased ; this is of great importance and, from this point of view, the maximum permissible tensile stress f_{tw} should ensure, if required, a definite factor of safety against cracking. It is impossible to combine great resilience with a high factor of safety against cracking. However, a high factor of safety is rarely required since, in the event of cracking, the cracks close on removal of the load. Consequently only in beams of type (i) subjected to impact is a large factor of safety against cracking necessary to avoid destruction of the bond by impact in the event of cracking.

With beams of type (ii) it is advisable to ensure that, under the total dead load or under loads that occur frequently, compressive stresses only occur, so that any cracks caused by a load that occurs occasionally will close. An unlimited nominal tensile stress should be allowed for beams of type (iii) only if there is no danger of fire, corrosion, or fatigue. In the writer's view, partially-prestressed beams of types (iB) and (ii) are preferable to fully-prestressed beams of type (iA). Absence of tensile stress is only necessary for beams subjected to impact. It is therefore possible to combine great resilience with permanent or temporary freedom from visible cracks and at the same time to reduce the cost compared with beams of type (iA). If the modulus of rupture is 1000 lb. per square inch, tensile stresses f_{tw} of 700 lb. per square inch should be permissible for constant loads and 900 lb. per square inch for occasional loads in beams of type (iB). Stresses of 1200 lb. per square inch (and even greater) can be used for beams of type (ii). Roof beams, for example, can be designed as type (ii) if the full working load occurs only once in many years, while a stress of 700 lb. to 900 lb. per square inch appears to be permissible if it occurs regularly.

The modulus of rupture depends on the shape of the cross section, the number and size of the holes for the cables, the size of the beam, and the quality of the concrete. Generally, for a concrete having a cube strength of 6000 lb. per square inch at 28 days, a modulus of 900 lb. to 1100 lb. per square inch is appropriate, provided that there is little interruption of the tensile zone and that the concrete there is well consolidated. Where the tensile zone does not comply with these provisions, a modulus of 700 lb. to 1000 lb. per square inch may apply.

(1) "Some New Developments in Prestressed Concrete," by P. W. Abeles. "The Structural Engineer," October, 1951.

Reduction of Prestress.

The magnitude of the reduction of the prestress should preferably be determined by tests but, since test results will not generally be available at the time the design is prepared, the most unfavourable stresses in the concrete should be considered, that is by assuming the smallest reduction at transfer and the greatest reduction at working load. To estimate these reductions the Report recommends that shrinkage may be assumed to be 0·03 per cent. for beams with pre-tensioned wires (one-third of this amount occurring at the time of transfer) and 0·02 per cent. for beams with post-tensioned wires or bars, provided that in beams with post-tensioned wires or bars the compressive force is applied to the concrete two to three weeks after the beam is cast. If the concrete in the plane of the tensioned wires or bars is subjected to a compressive stress of about 1000 lb. per square inch, the creep of the concrete may be assumed to be 0·04 per cent. for pre-tensioning and 0·03 per cent. for post-tensioning provided that in beams with post-tensioned wires or bars the compressive force is applied to the concrete two to three weeks after casting. The elastic deformation of the concrete may be calculated using modular ratios of 5, 6, and 8 if the cube strength of the concrete is 10,000 lb., 6000 lb., and 4000 lb. per square inch respectively. Creep of the steel may be counteracted by temporary overstressing.

Assuming the stress in the concrete adjacent to pre-tensioned steel to be 2000 lb. per square inch, corresponding to a maximum stress at the face of the beam of about 2400 lb. per square inch (which requires a cube strength of 6000 lb. per square inch at transfer), the total reduction is

$(6 \times 2000) + [0{\cdot}03 + (2 \times 0{\cdot}4)]10^{-2} \times 26{\cdot}5 \times 10^6 = 41{,}000$ lb. per square inch

if E_s is $26{\cdot}5 \times 10^6$ lb. per square inch. At transfer the reduction is

$$(6 \times 2000) + (0{\cdot}01 \times 10^{-2} \times 26{\cdot}5 \times 10^6) = 14{,}500 \text{ lb. per square inch.}$$

Assuming an initial tensioning stress p_i of 150,000 lb. per square inch, the effective prestress p_e is 109,000 lb. per square inch. Since it is advisable to consider the lowest estimate of reduction at transfer, a reduction of 10,000 lb. per square inch may be assumed; therefore p_t is 140,000 lb. per square inch and $R_o = \dfrac{109{,}000}{140{,}000} = 0{\cdot}78$, which agrees well with data obtained by measurement of strains.

With post-tensioned wires or bars, the amount of slip of the anchors and friction at tensioning must be known, and must be counteracted by increasing the tensioning force. The reduction of the initial prestress, with the same assumptions as before, is

$$[0{\cdot}02 + (2 \times 0{\cdot}03)]10^{-2} \times 26{\cdot}5 \times 10^6 = 21{,}200 \text{ lb. per square inch.}$$

Consequently R_o is 0·85 to 0·86 for a stress of 145,000 lb. to 150,000 lb. per square inch in the wires or bars at transfer, and is reduced to 0·79 if the stress at transfer is 100,000 lb. per square inch.

Minimum Moment of Resistance.

The formulæ for the general case given in this journal for April, 1948, can be restated as follows (*Fig.* 2):

$$\left.\begin{aligned} M_w - R_o M_s &= Z_1(R_o f_{1t} - f_{1w}) = Z_1(R_o f_{ct} + f_{tw}) = Z_1 f_{1m} \\ &= Z_2(f_{2w} - R_o f_{2t}) = Z_2(f_{cw} - R_o f_{2t(lim.)}) = Z_2 f_{2m} \end{aligned}\right\} \quad . \quad (1)$$

The term $R_o M_s$ must be omitted if the weight of the beam does not counteract the effect of the prestressing force, that is if the beam is not always the right way up and simply supported at its ends. If a beam is transported upside down while supported near the ends, or the right way up while suspended at the centre, the bending moment $-M_s$ must be inserted in formula (1), the first terms in which are then $M_w + R_o M_s$ and are independent of the sign of M_s. The bending moment M_s due to the weight of the beam, if always counteracting the effect of the prestressing force, is beneficial because generally no tensioned wires or bars

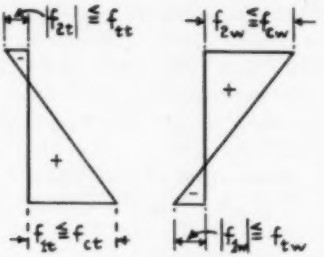


Fig. 2.

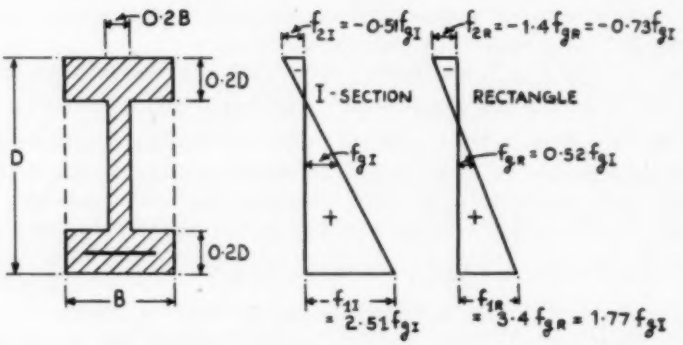


Fig. 3.

are required in the top of the beam and the depth of the beam can be less, but a greater tensioning force is required.

The stress $f_{2l(lim.)}$ is the limiting stress for a beam in which no tensioned wires are provided in the top, and is given by the formula

$$f_{2l(lim.)} = \frac{k_2}{k_1} f_{ct} + \frac{D}{k_1 I} M_s \quad . \quad . \quad . \quad . \quad . \quad (2)$$

Referring to *Fig.* 1, for a prestressing force P acting at a distance e_s from the centroid of the section, the stress at the bottom of the beam is $k_1 \frac{P}{A}$ and at the top $k_2 \frac{P}{A}$. The ratio $\frac{k_2}{k_1}$ is $\frac{r^2 - e_s e_2}{r^2 + e_s e_1}$; this is the ratio of the stresses at the bottom and

top and depends on the shape of the section. To make full use of the resistance of the concrete at the bottom and top of the beam, the adoption of an unsymmetrical section as in *Fig.* 1 is advisable. The dimensions of the section should be such that

$$\frac{Z_2}{Z_1} = \frac{e_1}{e_2} = \frac{R_o f_{ct} + f_{tw}}{f_{cw} - R_o f_{2t(lim.)}} = \frac{f_{1m}}{f_{2m}} \quad . \quad . \quad . \quad . \quad (3)$$

The stress $f_{2t(lim.)}$ may be tensile (that is negative) or compressive. If this stress is tensile and greater than the permissible stress f_{tt}, tensioned wires or bars are required in the top and $-f_{tt}$ must be substituted for $f_{2t(lim.)}$ in formulæ (1) and (3).

For a symmetrical I-beam (*Fig.* 3) the stress ratio $\frac{M_w - R_o M_s}{\alpha BD^2}$ is, from (1), either $\frac{R_o f_{ct} + f_{tw}}{6}$ or $\frac{f_{cw} - R_o f_{2t(lim.)}}{6}$. (For a rectangular section $\alpha = 1$.) If tensile stresses are not permitted and f_{ct} is 1000 lb. per square inch and R_o is 0·85, the moment of resistance is $141BD^2$, which is less than that of a reinforced concrete beam in which $t = 18{,}000$ lb. per square inch and $D = 1{\cdot}16d$. Therefore, by merely prestressing a beam, greater resistance cannot be obtained for the same stress in the concrete unless the weight of the beam counteracts the effect of the prestressing force. Only if the weight does counteract the effect of the prestressing force is the depth of a prestressed concrete beam in which tensile stresses are not permitted less than that of a reinforced concrete beam. Advantage is obtained if the prestress can be applied in stages so that not only the weight of the beam but successive additions of dead load can be made to counteract the effect of the prestressing force.

By increasing the permissible compressive stresses in the concrete the moment of resistance is considerably increased; for example, if the permissible stress at transfer is 2400 lb. per square inch, the moment of resistance of a rectangular beam is $340BD^2$. The writer has always advocated taking advantage of the great tensile resistance and resilience of prestressed concrete. If a tensile stress of 500 lb. to 600 lb. per square inch is permitted (which will ensure absence of cracks even under conditions of fatigue), the moment of resistance increases up to $440BD^2$. A bending tensile stress of 900 lb. per square inch, and consequently a moment of resistance of $490BD^2$, is permissible if the maximum load occurs rarely and permanent freedom from cracks is required. Temporary harmless cracks might occur if the tensile stress were 1200 lb. per square inch, the corresponding moment of resistance being $540BD^2$, which shows that greater moments of resistance are obtained by partial prestressing.

When designing a beam, the weight of the beam must be assumed first and is of greater influence if M_s is not deducted, since if M_s is deducted $(1 - R_o)M_s$ only is taken into account. If f_{tt} is to be substituted for $f_{2t(lim.)}$ in formula (3), the numerical value of f_{tt} can be used, but if $f_{2t(lim.)}$ applies a value must be assumed for this stress and corrected in a second calculation.

The stress ratios k_1, k_2, and k_0 depend on the shape of the cross section of the beam, and for a rectangular beam (*Fig.* 1) $k_1 = 3{\cdot}4$, $k_2 = -1{\cdot}4$, and $k_0 = -0{\cdot}411$ if the tensioned wires or bars are $0{\cdot}1D$ from the bottom or top of the beam. If the wires are in this position in a symmetrical I-beam having flanges $0{\cdot}2D$ deep (*Fig.* 3) and a web $0{\cdot}2B$ wide, the cross-sectional area is $0{\cdot}52BD$

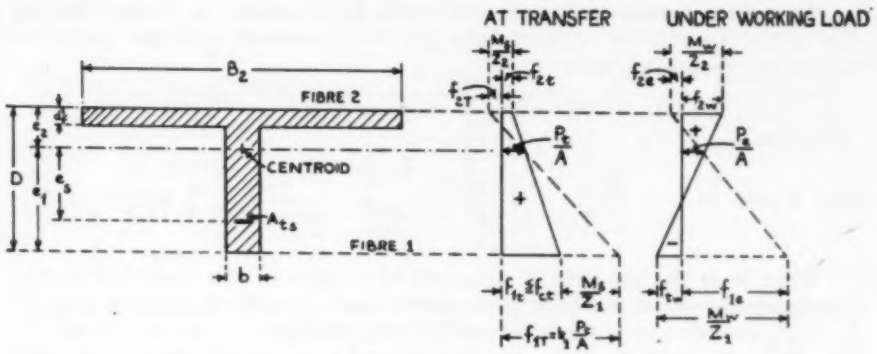
and the section modulus is $0{\cdot}827\frac{BD^2}{6}$; $k_1 = 2{\cdot}51$, $k_2 = -0{\cdot}51$, and $k_0 = -0{\cdot}203$. Therefore the compressive stress f_g at the centroidal plane is $\frac{P}{0{\cdot}52BD}$, and the maximum stresses f_1 and f_2 are $2{\cdot}51f_g$ and $-0{\cdot}51f_g$ respectively. In the rectangular end-block of this I-beam the compressive stress at the centroid is reduced to $0{\cdot}52f_g$ and the maximum stresses are $f_1 = 3{\cdot}4 \times 0{\cdot}52f_g = 1{\cdot}77f_g$ and $f_2 = -1{\cdot}4 \times 0{\cdot}52f_g = -0{\cdot}73f_g$. This calculation shows that, although the cross-sectional area of the rectangular section is nearly double that of the I-beam, the tensile stress f_2 is nearly 40 per cent. greater but the compressive stress is less. It is in most cases, however, important to obtain large compressive stresses f_1 and low tensile stresses f_2; hence rectangular beams are not very suitable.

When the weight of the beam counteracts the effect of the prestressing force immediately this force is applied, it is important to distinguish between the resultant maximum stresses (f_{1t} and f_{2t}) and the stresses due to the prestress alone (f_{1T} and f_{2T}), because the reduction of f_{1T} and f_{2T} must be considered, and not the reduction of f_{1t} and f_{2t}.

T-beams with Post-tensioned Wires or Bars.

When post-tensioning is applied, particularly to a cast-in-situ beam, and when economy and not the least depth of the beam is the more important factor, it is not the minimum section but the minimum prestressing force which is of greater importance because the cost of the cables, tensioning, anchoring, and grouting is generally the largest factor in the total cost. The counter-action of the weight of the beam allows smaller section moduli to be provided as is seen from formulæ (1), but the prestressing force required, depending on the prestress f_{1T}, is considerably increased as is seen in *Fig.* 4. Tensioned top wires or bars are not generally necessary since it is possible to bend up some of the cables towards the ends, where the weight of the beam does not counteract the effect of the prestressing force. In the central part of the span the cables should be placed as near the bottom of the beam as possible. As previously stated, the sectional area of the tensioned bottom cables, however, is much greater than that required if the weight does not counteract the effect of the prestressing force (in which case a deeper beam is required), since the stress f_{1T} and not the smaller stress f_{1t} governs the prestressing force P_t required. A T-beam is particularly suitable since k_1 is much greater than for beams of other sections, and the large compression flange is generally sufficient to provide the ultimate moment of resistance required.

If the minimum value of Z_1 is to be obtained the resultant stress f_{1t} must equal f_{ct}, and the prestress f_{1T} required is as given in *Fig.* 4, from which formula (4) in the diagram is derived. The example in *Fig.* 4 is a comparison of two cases, namely (*a*) a fully-prestressed beam designed for the minimum value of Z_1 ($f_{1t} = f_{ct}$), and (*b*) a partially prestressed beam in which a tensile stress of 800 lb. per square inch is permitted, and which is to have the same cross section and be designed to resist the same M_w as the beam in case (*a*). It is seen that the prestressing forces required in case (*a*) are considerably larger than those in case (*b*) for different ratios of $\frac{M_s}{M_w}$. The position of the tensioned wires is assumed to be the same in



REQUIRED PRESTRESSING FORCE $P_t = \frac{A}{R_1} f_{1T}$

REQUIRED PRESTRESS $f_{1T} = \frac{f_{1t}}{R_0} = \frac{1}{R_0}\left(\frac{M_w}{Z_1} - f_{tw}\right) = f_{ct} + \frac{M_s}{Z_1}$

$R_0\left(f_{ct} + \frac{M_s}{Z_1}\right) + f_{tw} = \frac{M_w}{Z_1}$, RESULTING IN $\frac{M_w}{Z_1} = \frac{R_0 f_{ct} + f_{tw}}{1 - R_0 \frac{M_s}{M_w}} = \frac{f_{1m}}{1 - R_0 \frac{M_s}{M_w}}$ . . . . (4)

EXAMPLE ($R_0 = 0{\cdot}85$)

CASE		$\frac{M_w}{M_s}$		4	2	1·33
a	$f_{1t} = f_{ct} = 2000$ LB/IN.^{2}; $f_{tw} = 0$; $f_{1m} = 1700$ LB/IN.^{2}	$\frac{M_w}{Z_1}$	LB/IN.^{2}	2160	2960	4670
		f_{1T}		2540	3480	5500
b	$f_{1t} < 2000$; $f_{tw} = 800$; $\frac{f_{tw}}{R_0} = 940$ LB/IN.^{2}; $\frac{M_w}{Z_1}$ AS CASE a.	f_{1T}		1600	2540	4560
	$\frac{P_t \text{ CASE (a)}}{P_t \text{ CASE (b)}} = \frac{f_{1T} \text{ CASE (a)}}{f_{1T} \text{ CASE (b)}}$			1·775	1·37	1·21

Fig. 4.—Cast-in-situ T-beams with Post-tensioned Wires.

both cases. However, reduction of the prestressing force results in a decrease in A_{ts}, and allows an increase of e_s; consequently k_1 increases, which means a further reduction of the prestressing forces required in case (*b*). In most cases there will be no need for additional untensioned reinforcement if the stress f_{1T} is relatively large, since the amount of A_{ts} required to obtain sufficient prestressing force will suffice for the ultimate moment of resistance.

I-beams Unsymmetrical about the Horizontal Axis.

Referring to *Fig.* 1, $\frac{Z_2}{Z_1} = \frac{e_1}{e_2} = \frac{D - 2\Delta e}{D + 2\Delta e}$ which, from formula (3), is equal to $\frac{f_{1m}}{f_{2m}}$.

Therefore $$\frac{\Delta e}{D} = \frac{1 - \frac{f_{1m}}{f_{2m}}}{2\left(1 + \frac{f_{1m}}{f_{2m}}\right)}.$$

Consider a symmetrical section of width B_0, as shown by broken lines in *Fig.* 1, and a section of the same area but unsymmetrical about the horizontal axis as shown by full lines.

$$[bD + 2(B_0 - b)d_f]\Delta e = (B_1 - B_2)d_f\left(\frac{D - d_f}{2}\right),$$

from which

$$\frac{B_1 - B_2}{B_0} = \frac{2\Delta e}{D} \cdot \frac{\frac{b}{B_0} + \frac{2d_f}{D}\left(1 - \frac{b}{B_0}\right)}{\frac{d_f}{D}\left(1 - \frac{d_f}{D}\right)}.$$

Since it is the moment of resistance that matters, the resistance of the unsymmetrical section will be larger than that of the symmetrical section because Z_1 or Z_2 exceeds the section modulus Z_0 of the symmetrical section. *Table* I[1] gives the ratio of Z_1 to Z_0 for the case of an I-beam in which $b = 0{\cdot}2B_0$ and $d_f = 0{\cdot}15D$ for various values of $\frac{f_{1m}}{f_{2m}}$ from $\frac{1}{2}$ to $1\frac{1}{2}$; $Z_0 = 0{\cdot}725\frac{B_0D^2}{6} = 0{\cdot}121B_0D^2$.

Consider, as an example, $\frac{f_{1m}}{f_{2m}} = 0{\cdot}8$; then $B_1 = 1{\cdot}19B_0$, $B_2 = 0{\cdot}81B_0$, and $Z_1 = 1{\cdot}1Z_0 = 0{\cdot}133B_0D^2 = \frac{M_w}{f_{1m}}$; or $B_0D^2 = \frac{M_w}{0{\cdot}133f_{1m}}$. If a value is assumed for D, then B_0 and consequently B_1 and B_2 can be evaluated; or, if B_0 is assumed, B_1, B_2, and D can be determined.

Design of Symmetrical I-beams with Pre-tensioned Wires.

The cost of precast prestressed concrete beams with pre-tensioned wires depends mainly on the volume of concrete required and to a less degree on the weight of steel necessary. (It is also affected by the number of beams required

TABLE I.—UNSYMMETRICAL I-BEAMS ($b = 0{\cdot}2B_0$; $d_f = 0{\cdot}15D$).

$\frac{f_{1m}}{f_{2m}}$	$\frac{\Delta e}{D}$	$\frac{B_1 - B_2}{B_0}$	$\frac{B_1}{B_0}$	$\frac{B_2}{B_0}$	$\frac{I}{B_0D^3}$	$\frac{Z_1}{B_0D^2}$	$\frac{Z_1}{Z_0}$	—
1·5	− 0·1000	− 0·6900	0·655	1·345	0·05612	0·0935	0·775	Unsymmetrical Section
1·4	− 0·0833	− 0·5748	0·713	1·287	0·05744	0·0985	0·815	
1·3	− 0·0652	− 0·4499	0·775	1·225	0·05867	0·1040	0·865	
1·2	− 0·0455	− 0·3140	0·843	1·157	0·05973	0·1092	0·905	$B_2 > B_1$
1·1	− 0·0238	− 0·1640	0·918	1·082	0·06026	0·1151	0·955	
1·0	0	0	1·000	1·000	0·06047	0·1209	1·000	Symmetrical Section $B_2 = B_1$
0·9	0·0263	0·1815	1·091	0·909	0·06026	0·1274	1·055	Unsymmetrical Section
0·8	0·0555	0·3830	1·191	0·809	0·05914	0·1330	1·100	
0·7	0·0884	0·6099	1·305	0·695	0·05698	0·1380	1·142	$B_1 > B_2$
0·6	0·1250	0·8625	1·431	0·569	0·05365	0·1420	1·175	
0·5	0·1667	1·1500	1·575	0·425	0·04830	0·1450	1·200	

(1) From "Further Notes on the Principles and Design of Prestressed Concrete," by P. W. Abeles. Civil Engineering and Public Works Review, April, 1951.

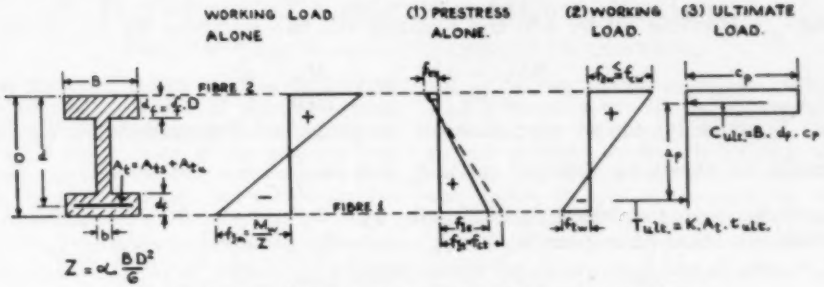


Fig. 5.—Symmetrical I-Beam with Pre-tensioned Wires (Minimum Section Modulus).

because the greater the number the more uses are obtained from the moulds.) It is therefore preferable if the beam has the smallest cross-sectional area, and a quick method of designing symmetrical I-beams is as follows.

It is assumed that the tensile flange is to be in accordance with the conditions described in the immediately preceding paragraphs. The compressive flange must comply with the requirements at ultimate load and, as shown in the article by the writer in this journal for October, 1951, conditions at ultimate load may require an increase of the size of the compressive flange and of the cross-sectional area of the steel by providing untensioned wires, that is to give $A_t = A_{ts} + K_u A_{tw}$. The stress diagrams are shown in *Fig.* 5. The effective prestress f_{1e} required is $R_0 f_{1t}$, which must equal $\frac{M_w}{Z} - f_{tw}$; therefore

$$\frac{M_w}{Z} = f_{1e} + f_{tw} = R_0 f_{ct} + f_{tw} = f_{1m} \quad . \quad . \quad . \quad \text{(i)}$$

which is identical with formula (1). For conditions at ultimate load,

$$M_w . FS = C_{ult} a_p = B d_f c_p (D - d_f) \quad . \quad . \quad . \quad \text{(ii)}$$

From (i), M_w is $\frac{\alpha}{6} BD^2 f_{1m}$, where α is the shape factor depending on $\delta_f \left(= \frac{d_f}{D}\right)$

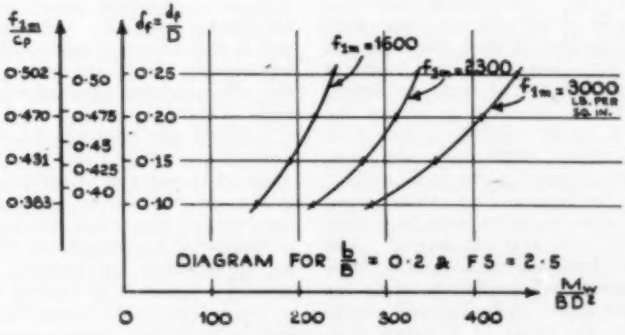


Fig. 6.

and $\frac{b}{B}$. Dividing (ii) by FS, and equating the two equations for M_w, gives

$$\frac{f_{1m}}{c_p} = \frac{6\delta_f(1-\delta_f)}{\alpha.FS}; \quad \text{and} \quad \frac{M_w}{BD^2} = \frac{\alpha}{6} f_{1m} \quad . \quad . \quad . \quad (5)$$

Formulæ (5) can be used to solve the problem. For example, in *Fig.* 6 curves are shown for different ratios δ_f, but for $\frac{b}{B} = 0{\cdot}2$ and $FS = 2{\cdot}5$. The three curves are for stresses f_{1m} of 1600 lb., 2300 lb., and 3000 lb. per square inch, which are obtained respectively for $f_{ct} = 2000$ lb. per square inch and $f_{tw} = 0$, $f_{ct} = 2000$ lb. and $f_{tw} = 700$ lb. per square inch, and $f_{ct} = 2500$ lb. and $f_{tw} = 1000$ lb. per square inch. In all cases $R_0 = 0{\cdot}8$. It is necessary to compute $\frac{f_{1m}}{c_p}$ only and the corresponding values of δ_f and $\frac{M_w}{BD^2}$. It is possible to prepare similar charts for other ratios $\frac{b}{B}$ and for other values of FS, and in each case for other values of f_{1m}. When such charts are available the method of designing the beam is not complicated. When substituting the numerical value of c_p, any residual stress in the top wires should be taken into account. Tensioned wires will generally be required in the top to allow the beam to be lifted in any position.

Stresses in Steel Reinforcement.

We have received the following from Mr. R. V. Chate, Secretary of the Reinforced Concrete Association.

The final paragraph of the Editorial Note in your February number suggests that there may be some difficulty in reconciling two points in the recommendation of the Reinforced Concrete Association regarding the permissible stress in mild steel reinforcement, which you published in January.

Please let me explain briefly that the Council of the Reinforced Concrete Association is of the opinion that, with the present quality of steel, the adoption of the higher stress for small diameter bars does not affect the factor of safety of a structure as a whole or reduce the factor for a given member below that for one in which larger bars are used, since the actual yield point and bonding qualities of the smaller bars are superior. When mild steel reinforcement bars with a sufficiently high, and guaranteed, yield point for all sizes become available, then the Association will probably press for the increased stress to be applied generally and for the Code of Practice (CP. 114—1948) to be amended accordingly.

The Council does not think that codes and standards need be altered to meet interim conditions, because such documents are not mandatory. CP. 114 was prepared for the guidance of chartered civil and structural engineers qualified in reinforced concrete, and they are at liberty to adopt its clauses or not at their discretion. Any amendment by the Council for Codes of Practice, even if the machinery at its disposal enabled it to make an amendment in time for it to be of practical value, would appear to confer upon the Code an authority which it does not possess and which it is undesirable, from the point of view of the advancement of reinforced concrete, that it should possess.

I would add that the London County Council has advised the Association that, while it is not the Council's practice to give a general waiver or modification of the by-laws to permit increased stresses, sympathetic consideration will be given to applications in respect of specific premises where it is desired to work to a tensile stress of 20,000 lb. per square inch in mild steel reinforcement bars not greater than $\frac{1}{2}$ in. diameter.

A Reinforced Concrete Office Building.

HOLLOW TILE SLABS AND CONCRETE CONTROL.

In the construction by the Ministry of Works of a reinforced concrete frame office building at Orpington, Kent, control of the making of the concrete was exercised with the object of economising in the use of cement and collecting data for future use.

The building is a three-story structure 34 ft. 6 in. high with single-story wings 14 ft. 6 in. high above the ground floor, below which there is a basement. The frontage is 134 ft., and the width varies from 38 ft. to 86 ft. The reinforced concrete frames are generally at 12 ft. centres. Some of the internal columns between the ground and first floors are circular and are 15 in. diameter; others are 14 in. by 12 in. rectangular columns. The external columns are either 12 in. square or 12 in. diameter. There are also some columns of composite shapes to suit architectural requirements. The circular columns were cast in shuttering constructed of sheets of stout cardboard bent to a circle and stiffened by vertical timber slats. All columns above the first floor are 12 in. square. Below the ground floor the interior columns are 18 in. by 14 in.; the exterior columns are 14 in. square and are 5 in. away from the basement retaining-walls, a position which simplified the construction of the foundation and made a heel to the retaining wall unnecessary.

The ground floor is a 6-in. solid reinforced concrete slab designed for a normal imposed load of 70 lb. per square foot, plus 55 lb. per square foot for finishes and future suspended heating pipes, plus 200 lb. per square foot for special loading. The maximum design load is therefore 325 lb. per square foot excluding the weight of the slab. The normal working stresses in bending are 1000 lb. per square inch in the concrete and 18,000 lb. per square inch in the reinforcement, but an increase of 25 per cent. is allowed when a debris load is included.

Hollow-Tile Slabs.

The first and second floors and roof are respectively 7½-in. and 6½-in. hollow clay-tile slabs (*Figs.* 1 and 2). Heating coils are embedded in 1½ in. of concrete below the tiles. The roof is designed for an imposed load of 30 lb. per square foot plus 44 lb. per square foot for finishes, heating coils, and the concrete in which the heating coils are embedded. The upper floors are designed for an imposed load of 70 lb. per square foot, which includes for light partitions, plus 55 lb. per square foot for finishes, heating coils, etc. Partitions of 4-in. slabs in the same direction as the span of the slabs are carried on cast-in-situ beams. Parts of the floors to be used as stores are solid slabs designed for an imposed load of 200 lb. per square foot.

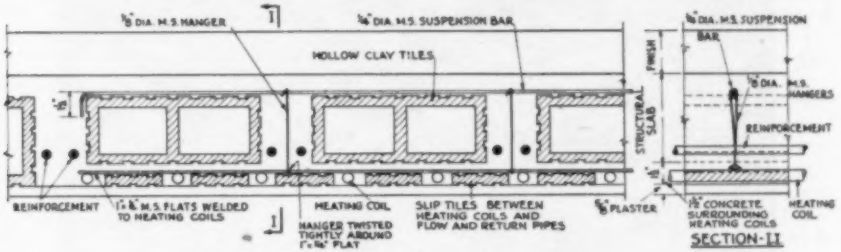


Fig. 1.—Details of Floor Construction.

The method of construction of the hollow-tile slabs was to lay the coils on steel shuttering and fix the reinforcement. Fluted slip-tiles were laid on the shuttering between the coils to form a key for ⅝ in. of ceiling plaster. The coils were anchored by ⅛-in. mild steel wires which are hung from a ¼-in. bar lying on the hollow clay-tiles and which are twisted around 1-in. by ¼-in. mild steel flats

welded across the coils. Concrete was placed first between and over the slip-tiles to a depth of about 1½ in. The hollow tiles, which are 12 in. by 12 in. by 3 in. or 4 in., are keyed on all faces, and were soaked in water and laid immediately on the layer of wet concrete. Concrete was then placed between and over the tiles to the required total thickness of the slab.

Fig. 2.—Slip Tiles Between Heating Coils.

Concrete Control.

The concrete was weigh-batched in the hopper of a 10/7 non-tilting drum mixer to which a weighing scale was attached. The mixture is nominally 1 : 2 : 4 generally and in the hollow-tile slabs, where by careful proportioning and mixing a concrete of sufficient strength and workability was obtained without increasing the proportion of cement. The cement is ordinary Portland. The coarse aggregate is washed crushed gravel and was supplied in two sizes, namely, ⅜ in. to ⅛ in. and ⅞ in. to $\frac{5}{16}$ in. Generally aggregate of the two sizes was used in suitable proportions, but in the hollow-tile slabs the smaller size only was used. The fine aggregate is $\frac{5}{16}$-in. sand containing a large proportion of smaller grains but was considered satisfactory when combined with the coarse aggregates. The sand was obtained from a pit at a reasonable distance from the site, and there was no justification for obtaining a coarser sand from a greater distance and at extra cost.

In general, the mixture for reinforced concrete, except for the hollow-tile slabs, was 112 lb. of cement, 250 lb. of sand, and 470 lb. of combined ¾-in. and ⅜-in. coarse aggregates. The weights are those

TABLE I.—STRENGTHS OF CONCRETE.

Size of gravel aggregate	Quantity (lb.) of dry aggregates per cwt. of cement		Water-cement ratio	Average breaking load (lb.) of beams at 3 days	Average crushing strength (lb. per square inch) of cubes	
	Gravel	Sand			at 7 days	at 28 days
¾ in.	470	250	0·6	1390	2780	4200
⅜ in.	455	265	0·7	1390	2320	3460

in one batch assuming dry aggregates, and were altered to allow for the moisture content in the aggregates. Taking into account the specific gravity of the materials, the mixture is very nearly 1 : 2 : 4 by volume. The water-cement ratio was maintained at 0·6. Sieve analyses were made regularly at the site to check the grading of the fine and coarse aggregates. The water content of the sand was measured each day.

The concrete in the hollow-tile slabs is made with ¾-in. gravel and had a water-cement ratio of 0·7. The consistency was suitable for placing and ramming by hand the concrete surrounding the heating coils and in the ribs and topping. Each batch of concrete contained 112 lb. of cement, 265 lb. of sand, and 455 lb. of ¾-in. gravel, the sum of the weights of the sand and coarse aggregate being the same as for the concrete made with the larger aggregate.

Tests were made at three days on 4-in. square beams 16 in. long, and on 6-in. cubes at seven and 28 days. The beams were tested at the site in the Ministry of Works standard beam-testing machine. A central load is applied, in increments until the beam breaks, at the centre of the beam, which is supported at 12 in. centres. Some average results for the concrete used are given in *Table* I.

The building was designed by the Chief Architect's Division of the Ministry of Works, the senior architect being Mr. E. H. Banks, F.R.I.B.A., F.R.S.A., and is being constructed under the supervision of the architectural and structural departments. The structural engineer is Mr. G. H. Stewart, M.I.Struct.E., of the Ministry of Works, in association with the nominated consulting engineer, Mr. T. Bedford, M.Inst.C.E., who designed the reinforced concrete work. The contractors are Messrs. John Morgan (London), Ltd.

Fish Passes at Dams.

IT is known that large fish will readily travel upstream at the sites of dams by means of the usual ladder-like fish-pass, or, in the case of dams of exceptional height, by attracting the fish up a fish-pass to a tank in which they can be elevated to the top.

There is, however, great mortality among the small fish travelling downstream to the sea which pass through the turbines. In the case of two new dams to be built in the Cowlitz river in the state of Washington, and which it is thought would seriously affect the fishing below the dams, a proposal has been made to pass the small fish downstream through a pipe. The small fish would be collected in a chamber of 12 ft. diameter above the dam and prevented from leaving the chamber by a screen at the lower end. Decompression of the fish would be effected by closing the intake valve and allowing the water to drain from the bottom at a predetermined rate, after which the fish would pass through a pipe to the stream below. The reason for the decompression of the fish is that it was found that fish are killed by rapid decompression in passing from a penstock to a draught tube.

Prestressed Concrete Bridges in Germany.

THREE prestressed concrete bridges were built last year in Schleswig-Holstein. The bridge over the railway near Eckernförde (*Fig.* 1) is built on a skew of 43 deg. and is prestressed in two directions on the Freyssinet system. The end spans are 38 ft. long and the middle span is 45 ft. long. The bridge over the river near Eckernförde (*Fig.* 2) has end spans of 30 ft. and a middle span of 36 ft. It is on a skew of 70 deg. and is prestressed in the longitudinal direction only on the Leonhard-Bauer system. Strain measurements have been carried out on both these bridges by Dr.-Ing. Helmut Vogt, of Eckernförde, and it is expected that the results of these measurements will be published.

The bridge shown in *Fig.* 3 is near Kiel, and is on a skew of 43 deg. The span is about 86 ft. It is claimed that this type of construction, in which only the ribs are prestressed (on the Magnel system), is very economical.

Fig. 1.

Fig. 2.

Fig. 3.

The Main Access Tunnel at London Airport.

THE main access tunnel now in course of construction at London Airport is 2041 ft. long, excluding approaches, and about 86 ft. wide. The tunnel (*Fig.* 2) is separated by longitudinal walls into two outer divisions for pedestrians and cyclists and two divisions for vehicles; each of the divisions for vehicles has a 20 ft. carriageway for two lines of vehicles. The cycle and pedestrian paths are raised above the general level of the bottom of the tunnel, and below them are ventilation ducts. The structure is of reinforced concrete and is being constructed in an open excavation (*Fig.* 1). The machine seen in the right-hand foreground of *Fig.* 1 is used for driving piles at the toe of the sloping sides to prevent the soil from slipping into the excavation. The tunnel extends from the central area of the airport, where terminal buildings are to be erected, towards an entrance on the Bath road, and passes below three taxi-cab tracks and a main runway. There will be about 10 ft. of cover above the tunnel.

The floor of the tunnel (*Fig.* 3) is a reinforced concrete slab 15 in. thick under the roads and 30 in. thick under the walls. The outer walls are 18 in. thick and the intermediate walls 14 in. thick. The roof slab, which is 2 ft. 4 in. thick, is monolithic with the walls. Hinges are formed in the thickness of the walls at the junctions with the floor slab. The tunnel is divided into lengths of 50 ft. by transverse contraction joints extending through the walls, floor, and roof. The joints are simple butt joints made watertight by a strip of flexible copper crossing the joint.

The excavation is about 30 ft. deep and is through a thin layer of top soil, about 4 ft. of brick earth, and 7 ft. to 8 ft. of water-bearing gravel, and extends into blue clay below. The sides of the excavation slope at 1 in 1½. A protective layer of concrete 3 in. thick is deposited

Fig. 1.—View during Construction.

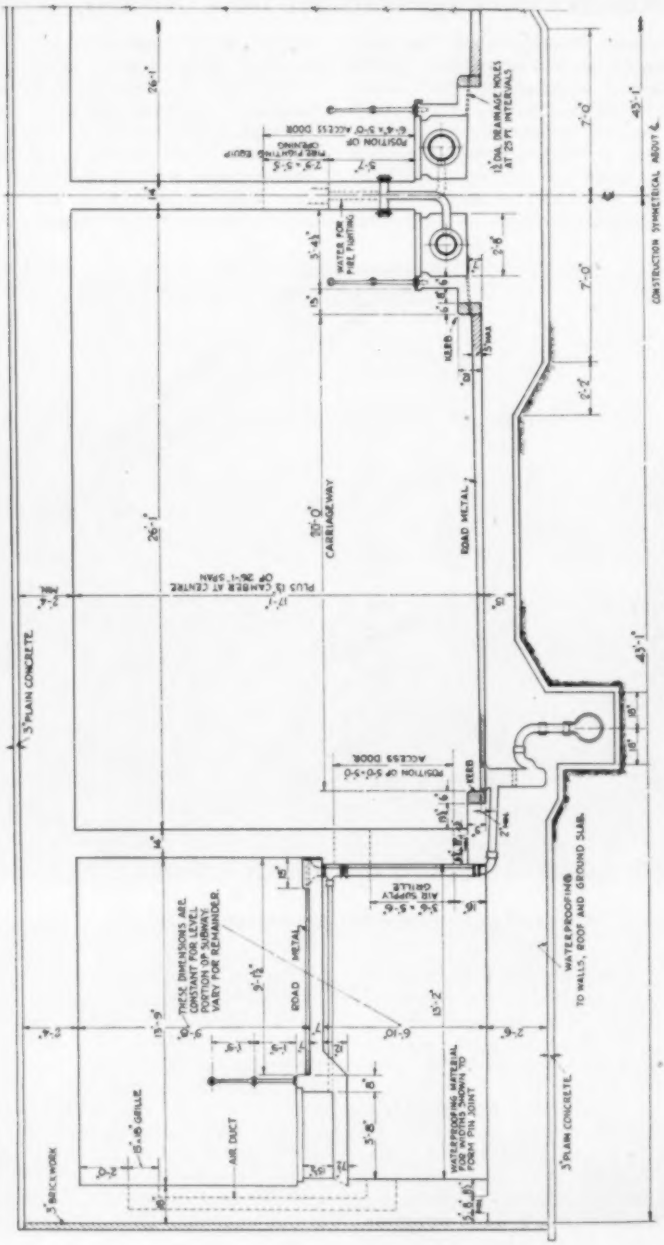

CONSTRUCTION SYMMETRICAL ABOUT ℄
12" DIA. DRAINAGE HOLES AT 25 FT. INTERVALS
POSITION OF PIPE FOUNTAIN EQUIP. (OPENING) 6'-6" x 5'-0" ACCESS DOOR
WATER FOR PIPE FOUNTAIN
KERB
CARRIAGE WAY
ROAD METAL
PLUS ½ CAMBER AT CENTRE OF 26'-1" SPAN
3" PLAIN CONCRETE
POSITION OF 5'-0"x5'-0" ACCESS DOOR
AIR SUPPLY GRILLE
THESE DIMENSIONS ARE CONSTANT FOR LEVEL PORTION OF SUBWAY. VARY FOR REMAINDER.
WATERPROOFING TO WALLS, ROOF AND GROUND SLAB
3" BRICKWORK
AIR DUCT
WATERPROOFING MATERIAL TURNED DOWN TO FORM PIN JOINT
3" PLAIN CONCRETE

Fig. 2.—Cross Section of Tunnel.

on the clay. Two layers of bituminous sheeting bonded with hot bitumen are laid on the concrete and covered with a further protective layer of plain concrete 2 in. thick. The sheeting extends up the outer face of the outer walls and over the top of the roof, thereby totally enclosing the tunnel in a watertight membrane. For a height equal to the depth of the floor of the tunnel, the vertical sheeting is laid against an outer 4½-in. skin of brickwork and the concrete in the floor is placed against the sheeting, the top of which is temporarily bent outwards. When the walls have been constructed, the sheeting is bent back into a vertical position and laid against and extended up the walls, and is protected on the

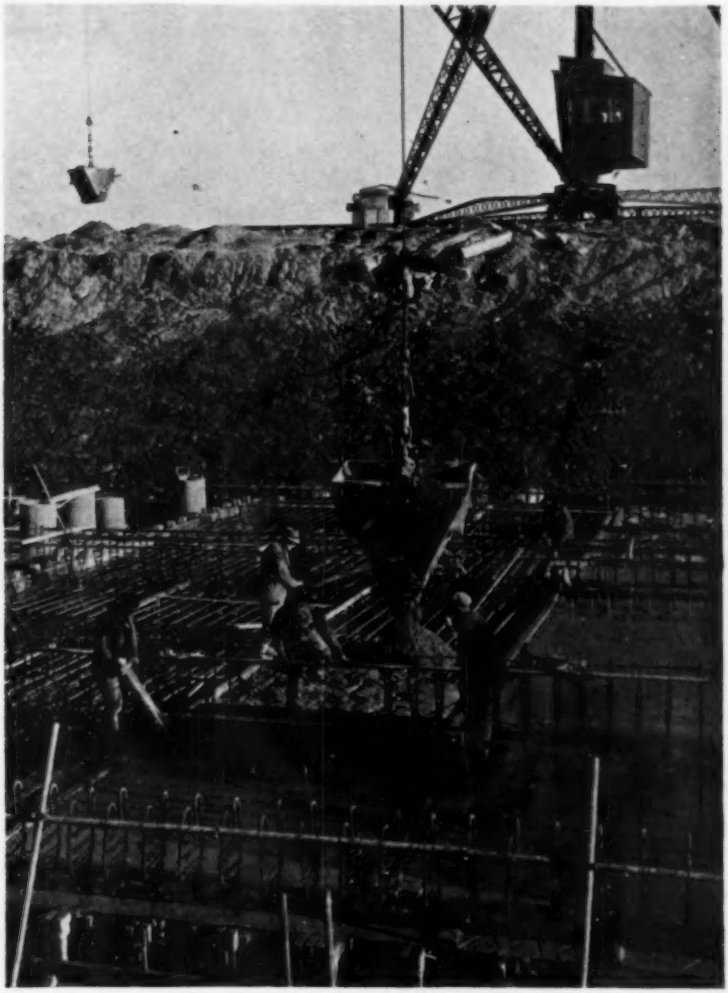


Fig. 3.—Concreting the Floor.

outside by concrete blocks. The sheeting on the roof is protected by a layer of plain concrete 3 in. thick.

After the excavation has been done, two longitudinal collector drains for stormwater from the road gullies are laid below the level of the 3-in. protective layer and surrounded by concrete. The construction of the tunnel then proceeds as follows. (1) The watertight sub-base is laid on the bottom of the excavation. (2) The reinforced concrete floor slab is laid to its entire depth in lengths of 50 ft.; the two narrow strips of bituminous sheeting in each of the hinges at the bottom of the walls are laid, and additional concrete is laid to a depth of 2 in. above the sheeting for the entire width of each wall; the hinge bars project through this concrete. (3) The walls are concreted in lengths of 25 ft. for the entire height of about 17 ft. in one operation; between successive lengths the joints are alternately construction and contraction joints. (4) The roof is constructed in lengths of 50 ft. for the entire width of the tunnel.

Along the top of each slope of the excavation there is a 7-tons derrick crane travelling on double tracks (*Fig.* 4). Concrete is brought in batches of 2 cu. yd. in motor lorries from a central batching plant and is discharged into a bottom-opening skip, which is taken by the cranes to the work (*Fig.* 3). The concrete has a water-cement ratio of 0·62 and is consolidated by high-frequency vibrators. The proportions of cement to aggregate are adjusted to produce a concrete that has a crushing strength of not less than 2850 lb. per square inch at 28 days. The shutters for the walls and roof of the tunnel are in 25-ft. lengths and are suspended from steel travellers running on rails on the floor of the tunnel. There are three travellers for the walls. The traveller for the central wall runs on a rail on either side of the wall, the shutters being suspended from a crosshead (*Fig.* 5). The traveller for one outer wall and the wall between the adjacent pedestrian and the vehicular lines runs on one rail in the pedestrian line and one in the vehicular line and carries, suspended from the crosshead, the shutters for the inner face of the outer wall and both faces of the division wall. The shutter for the outer face of the outer wall comprises $\frac{1}{16}$-in.

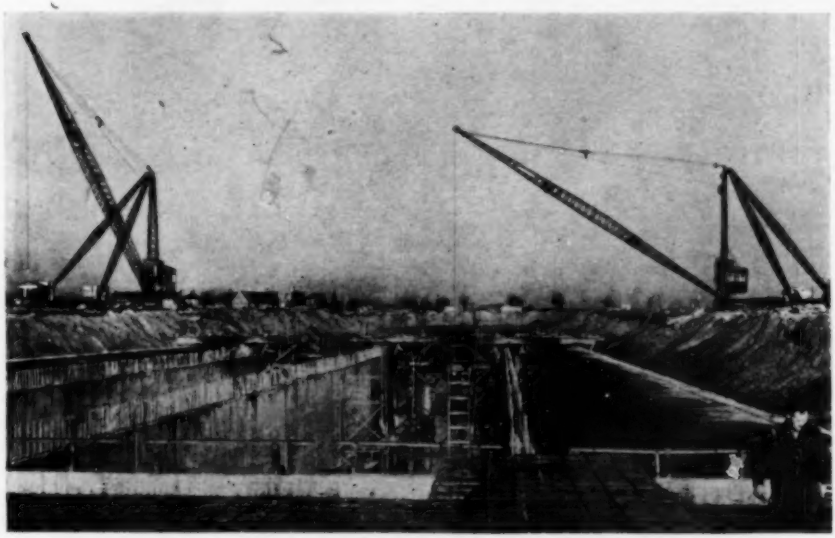


Fig. 4.—Cranes Used for Placing Concrete.

STEEL FORMS

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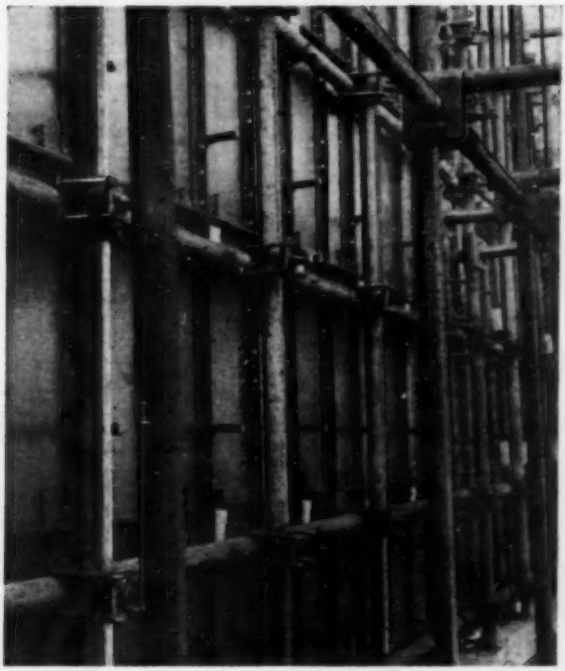
STEEL SHUTTERING

for in-situ concrete construction



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steel plate welded to vertical 12-in. by 5-in. steel beams at 24½-in. centres, and is suspended from a cantilevered extension of the crosshead.

The travellers carrying the roof shutters, which are in 25-ft. lengths, are each supported on two rails on the floor of the tunnel, and have a wheelbase of 15 ft. In the cross section of the tunnel there are ten such travellers, namely, two in each pedestrian line and three in each vehicular line. The travellers are used in pairs in tandem so that a 50-ft. length of roof can be constructed at one time. Each traveller supports transverse timber joists and 2-in. boarding. Between the boarding carried by adjacent travellers there is a longitudinal wooden strip shutter which is not carried on the travellers but is supported on 6-in. diameter tubular steel posts bearing on the floor. Each post is fitted with a small screw-jack for vertical adjustment. Four days after casting each 50-ft. section of the roof slab, the main shutters are lowered slightly on screw-jacks on top of the travellers, which are then moved forward to the next position. The strip shutters remain in position supported by the posts until the concrete is sufficiently strong not to need support.

The tunnel is generally level throughout, but at the ends there are inclines and the construction of the shutters allows for the formation of the vertical curves. The entrance to the tunnel is of variable

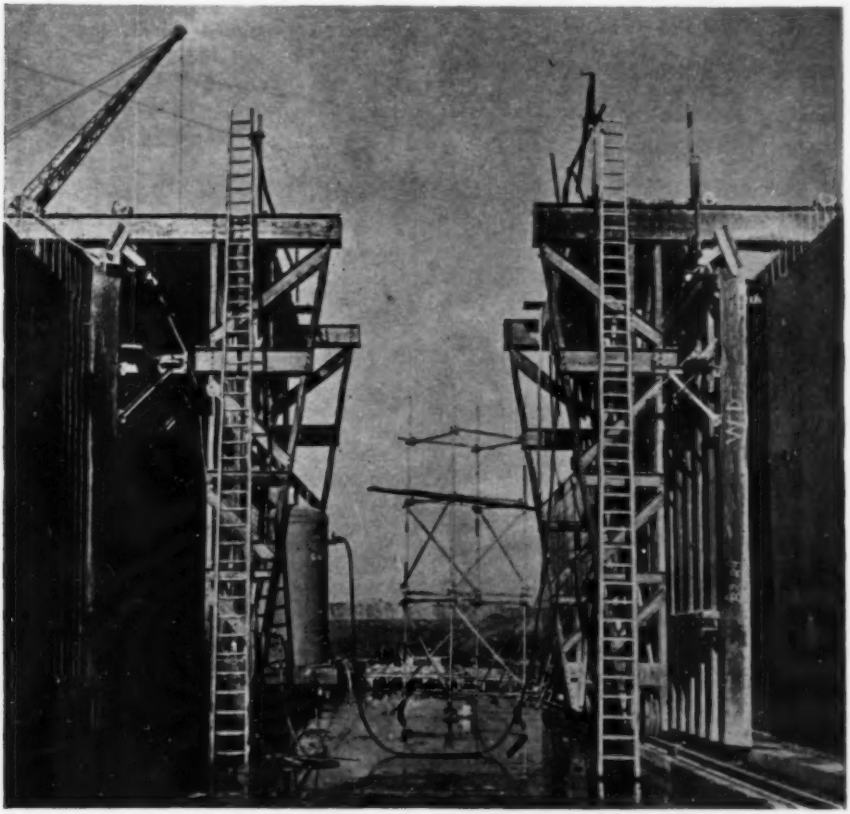

Fig. 5.—Shuttering for Dividing Walls.

cross section and will be constructed later, using timber shuttering.

The normal rate of construction is 50 ft. weekly, and depends on the time which must elapse before the roof shuttering can be released. Excavation commenced in November 1950, and it is expected that the tunnel will be completed by the end of 1952. There are about 750,000 cu. yd. of excavation, nearly 100,000 cu. yd. of plain and reinforced concrete, and 3000 tons of reinforcement in the tunnel, including the approaches and ancillary works. The tunnel was designed by the Air Ministry Works Directorate for the Ministry of Civil Aviation, and the contractors are Taylor Woodrow Construction, Ltd.

New Method of Building Multi-story Structures.

THE accompanying drawings show the design of blocks of residential flats, from four to ten stories in height, to be built at Clydebank. A total of 1700 dwellings will be provided. The consulting architects are Messrs. Sam Bunton and Associates and the consulting engineers Messrs. F. A. MacDonald and Partners. The cost per dwelling is estimated to be £1720 compared with £2000 for a separate dwelling with the same accommodation. The exterior walls and the transverse walls are load bearing and are built with concrete blocks which also serve as shuttering and the floors are built with prestressed concrete planks, so that no shuttering is required. The wall blocks comprise two slabs 18 in. long by 9 in. high by 2 in. thick, spaced 4 in. apart and joined by reinforced concrete ties (*Fig.* 1). When three or four courses of blocks have been laid light reinforcement is placed in the cavity and the concrete placed. In this way the walls are carried up to each floor level, where prestressed concrete planks 6 in. wide by 2 in. thick are laid on the transverse walls with a bearing of about 1¾ in. at each end. Nailable strips for fixing plaster-board are formed on the top edge of the inner faces of the blocks, so that there is an air-space between the concrete and the plaster-board.

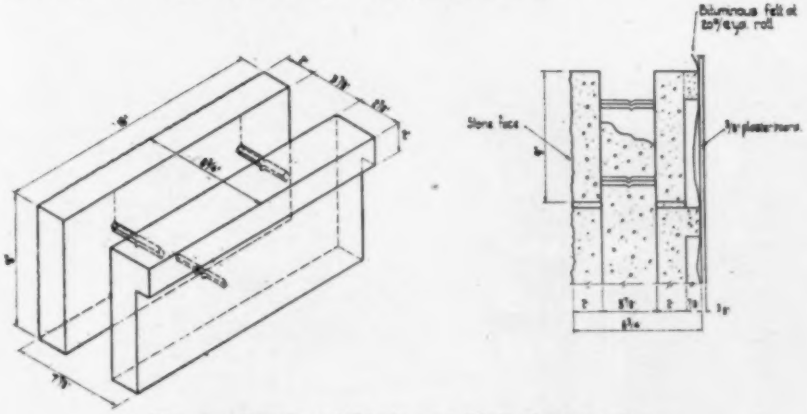

Fig. 1.—Details of Block and Wall Construction.

The prestressed concrete slabs, which span up to 18 ft., have interlocking edges (*Fig.* 2), and are supported after they are laid in position by a steel joist propped transversely under the middle of each panel of slabs in those cases only where the span is more than 14 ft. When the planks are in position, light reinforcement is laid over the planks and cross-walls, and a 1-in. concrete topping is placed. The upper faces of the planks are grooved in order to ensure a good bond with the topping. A feature of the design

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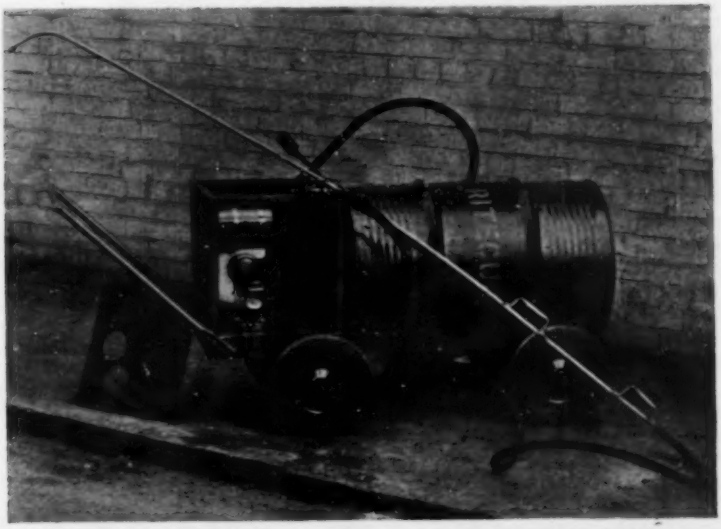

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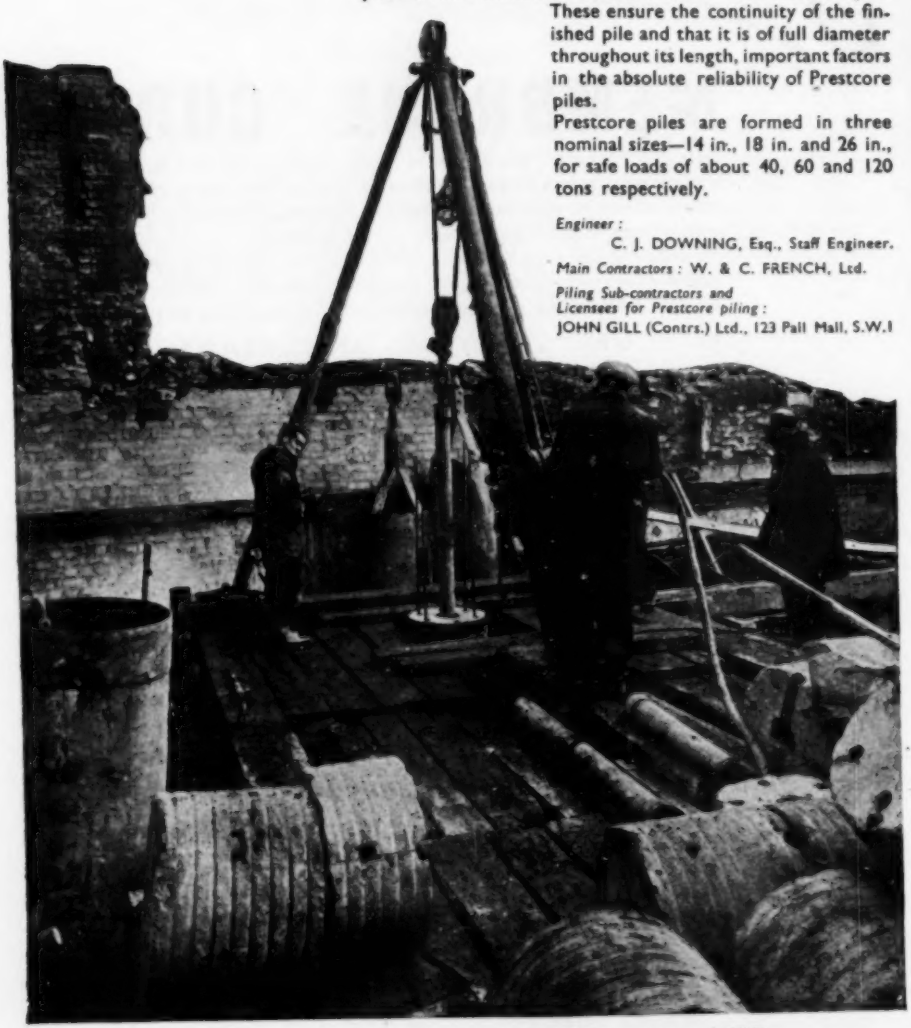

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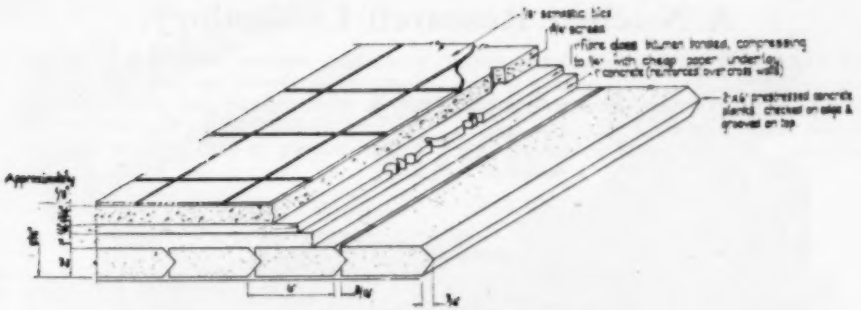

Fig. 2.—Floor Construction.

of the planks spanning more than 18 ft. is that the props are adjusted so that the middle of the planks, which sag when laid in position, is the same height as the ends, so that the amount of precompression is reduced because the upper face of the middle third of the planks is in a state of tension, while the upper face of the end thirds is in compression. When the props are removed the tendency of the middle of the planks to sag is resisted by the concrete topping. The result is to reduce the tensile stresses in the upper surface of the topping. The thickness of the floor is 3 in. only, and this effects a saving of 4 ft. in the height of a ten-story building compared with the type of floor commonly used in residential flats.

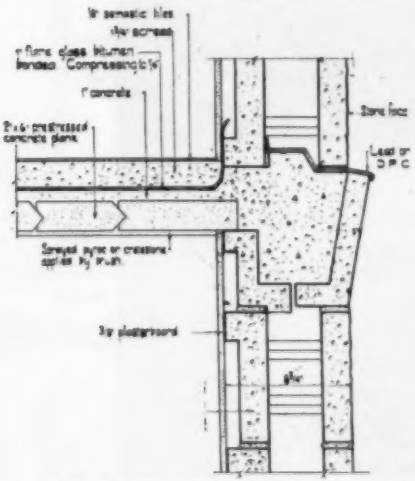

Fig. 3.—Junction of Floor and External Wall.

Some of the features of this design are patented, and the system is controlled in Great Britain by Multi Conflats, Ltd., of Glasgow.

British Standard for Hollow Clay Blocks.

REVISED British Standard No. 1190 (1951), "Hollow Clay Building Blocks" (price 2*s.* from the British Standards Institution) deals with blocks for internal load-carrying walls, partitions, and structural slabs. Wall-blocks may have faces keyed for plaster, both faces smooth for use without plaster, or keyed on one face and smooth on the other. Tests of crushing strength, water absorption, and shape are described. The dimensions of blocks for various uses are given. Standard wall-blocks are 12 in. long and 8⅛ in. high, or 9⅛ in. high if required to bond with larger bricks. The crushing strengths of blocks for partitions must be not less than 200 lb. per square inch, and for load-carrying walls 500 lb. per square inch. Blocks for structural slabs are 12 in. long, 10 in. or 12 in. wide, and 3 in., 3½ in., 4 in., 4½ in., 5 in., 6 in., 7 in., or 8 in. deep; the crushing strength must be not less than 2500 lb. per square inch.

Precast Concrete Floors.

A REVISED edition of the Bison Information Book has been issued by Concrete, Ltd (Stourton, Leeds 10), and copies will be sent free of charge to those concerned with precast concrete floors. The book, which comprises 148 pages, contains a great deal of useful information on the properties of precast floor beams and methods of erection.

A Nuclear Research Laboratory.

IN *Fig.* 1 is shown the building recently completed at Liverpool University to contain a 156-in. cyclotron in connection with a new laboratory for research in nuclear physics. The concrete is designed to give protection against radiations from the neutrons produced by the collision between atomic nuclei and protons of four hundred million volts. The architect for the laboratory is Mr. William Holford,

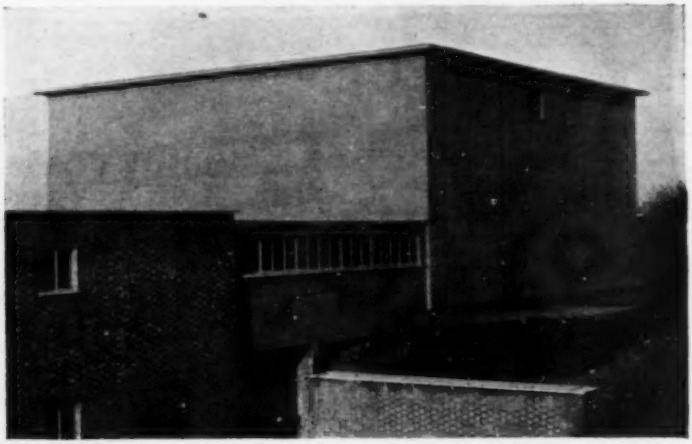

Fig. 1.—Building Housing the Cyclotron.

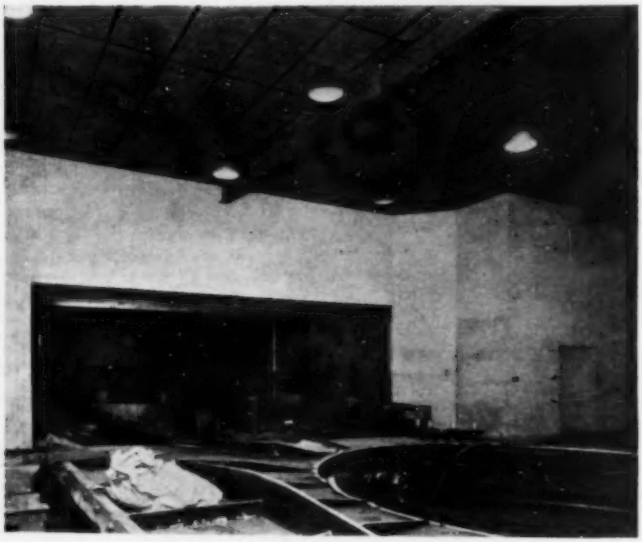

Fig. 2.—The Cyclotron Chamber during Construction.

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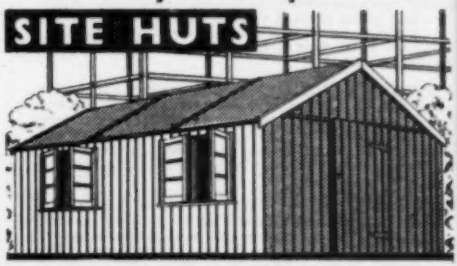

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MISCELLANEOUS ADVERTISEMENTS.

Situations Wanted, 3d. a word : minimum 7s. 6d. Situations Vacant, 4d. a word : minimum 10s. Other miscellaneous advertisements, 4d. a word : 10s. minimum. Box number 1s. extra. The engagement of persons answering these advertisements is subject to the Notification of Vacancies Order, 1952.

Advertisements must reach this office by the 23rd of the month preceding publication.

SITUATIONS VACANT.

SITUATIONS VACANT. Structural engineering designer/draughtsmen required in Designs Branch by Air Ministry Works Department. Applicants should have had several years' experience in the design and detailing of reinforced concrete or structural steelwork. The appointments will normally be in London. Salaries are in ranges up to £675 per annum with starting pay dependent upon age, qualifications and experience. Applications, stating age, qualifications, previous appointments (with dates), should be sent to AIR MINISTRY DIRECTORATE GENERAL OF WORKS, C.22 (W.9), Bush House, S.E. Wing, Strand, London, W.C.2, from which address further details may be obtained.

SITUATIONS VACANT. Reinforced concrete draughtsmen. Detailers and Junior designers required for long-term oil refinery and chemical plant projects. Pay commensurate with ability. Five-day week, pension, and insurance plans. Write, stating experience and salary required, to CHIEF ENGINEER, E. B. Badger & Sons (G.B.), Ltd., 40 Parkgate Road, London, S.W.11.

SITUATION VACANT. Design engineer required by consultant for prestressed concrete and reinforced concrete. Degree, and at least three years' reinforced concrete design experience, desirable. Work in London. Mainly bridges and buildings, with occasional site visits. Alternate Saturdays off. Permanent and progressive. DONOVAN H. LEE, M.I.C.E., 66 Victoria Street, London, S.W.1.

SITUATIONS VACANT. Reinforced concrete designers and draughtsmen required for progressive positions. Apply by letter giving training, experience, and salary required, to LESLIE TURNER AND PARTNERS, Consulting Engineers, 6 Grove Road, Sutton, Surrey, or 116 Victoria Street, London, S.W.1.

SITUATIONS VACANT. Detailer draughtsmen with experience of reinforced concrete work urgently required by consulting engineers at Bayswater. Permanent positions, good salary, and prospects. Apply with full details to CLARKE, NICHOLLS & MARCEL, 21 Westbourne Grove, London, W.2, or telephone Bayswater 6816 for appointment.

SITUATION VACANT. Contract manager required for control of reinforced concrete construction on building sites. Should have knowledge of site organisation, granolithic floor laying, and reinforced concrete design. Age under 40. Working from head office. Applicants to state experience and salary required. Address applications to C. J. TATHAM, Commercial Manager, John Ellis & Sons, Ltd., 21 New Walk, Leicester.

SITUATION VACANT. Civil engineer required by British firm of civil engineers and contractors, with head office at Singapore, to undertake preparation of design for important water supply scheme. Previous experience of this class of work is essential. Write with full particulars to BOX FE/65, c/o 95 Bishopsgate, London, E.C.2.

SITUATIONS VACANT. Imperial Chemical Industries Limited, Wilton Works, near Redcar, requires to augment its present Drawing Office staff for the development of its large new chemical factory. Permanent employment will be offered to selected Draughtsmen in the following categories : Civil, Instrument Installation, Mechanical, Structural. Congenial working conditions include membership of staff pension fund, recreation club facilities, etc. Further details of conditions of employment, pension fund, and housing will be forwarded on request, together with official application for employment form. Apply to the STAFF OFFICER, IMPERIAL CHEMICAL INDUSTRIES LIMITED, Wilton Works, P.O. Box 54, Middlesbrough, quoting advertisement reference ICI/X/106/i.

SITUATIONS VACANT. The British Reinforced Concrete Engineering Co., Ltd., require several qualified designers with specialist experience for their Stafford, London, Bristol, Glasgow, Dublin, and Newcastle-upon-Tyne offices. Five-day week and staff pension scheme. Apply to B.R.C. ENGINEERING CO., LTD., Stafford.

SITUATION VACANT. Senior reinforced concrete designer wanted in the Midlands area. House may be provided for suitable applicant. Reply stating salary required to BOX 2539, CONCRETE AND CONSTRUCTIONAL ENGINEERING, 14 Dartmouth Street, London, S.W.1.

SITUATIONS VACANT. Detailer-draughtsmen and one designer-draughtsman urgently required by consulting engineers. When replying please state age, experience, and salary required. Permanent position. Apply in writing to MR. O. MARCEL, MESSRS. CLARK, NICHOLLS & MARCEL, 21 Westbourne Grove, London, W.2. Bayswater 6816.

SITUATIONS VACANT. Reinforced concrete designers wanted for Southern Rhodesia and Union of South Africa. Candidates should have had at least 5 years' experience of competitive designing. Free passages. Salary according to experience and qualifications. Details in confidence to BOX R13, c/o 95 Bishopsgate, London, E.C.2.

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SITUATION VACANT. Young engineer required for Midlands, with knowledge of reinforced concrete or structural steelwork, to follow up sales inquiries for structures in a proprietary method of building construction. SECRETARY, TARSLAG, LTD., WOLVERHAMPTON.

SITUATIONS VACANT. The Trussed Concrete Steel Co. Ltd., Truscon House, 35–41 Lower Marsh, London, S.E.1, require additional staff in their London and Manchester design offices : (*a*) Designer-Detailers (age 25–45), (*b*) Detailer-Draughtsmen (age 20–35). Previous good experience in reinforced concrete drawing office is important. Five-day week and pension scheme. Apply in writing to the above address giving full particulars of age, education, and previous employment.

SITUATIONS WANTED.

SITUATION WANTED. B.Sc., D.I.C., A.M.I.Struct.E., experienced in reinforced concrete and steelwork design and specialising in prestressed concrete and shell roofs, requires London appointment with progressive consultants or contractors for design and site supervision. Available about August. BOX 2537, CONCRETE AND CONSTRUCTIONAL ENGINEERING, 14 Dartmouth Street, London, S.W.1.

SITUATION WANTED. Reinforced concrete detailer, two years' experience, H.N.D. Graduate I.Struct.E., taking A.M.I.Struct.E. examination in July, requires a position which would provide him with experience in reinforced and prestressed concrete design. BOX 2538, CONCRETE AND CONSTRUCTIONAL ENGINEERING, 14 Dartmouth Street, London, S.W.1.

FOR SALE.

FOR SALE. Sacks, bags, and curing cloths for sale. You want the best type and quickest delivery. Write JOHN BRAYDON, LTD., 26 The Highway, London, E.1. Telephone : ROYal 1044.

FOR SALE. Mobile concrete breaker compressor. Armstrong-Whitworth, 3 tool. Direct coupled to Dorman petrol engine. Mounted on Dennis lorry chassis, with enclosed cab. Photo from F. J. EDWARDS, LTD., 359 Euston Road, London, N.W.1. EUSton 4681.

PATENTS.

PATENT. The proprietors of British Patent No. 606232 for IMPROVED JETTIES AND SIMILAR STRUCTURES IN CONCRETE, AND A METHOD OF CONSTRUCTION THEREOF, desire to enter into negotiations with a firm or firms for the sale of the patent or for the grant of licences thereunder. Further particulars may be obtained from MARKS & CLERK, 57 & 58, Lincoln's Inn Fields, London, W.C.2.

PATENTS. British rights in a series of four British patents relating to improvements in prestressed concrete beams, slabs and other units for sale outright. Specifications and details from the PATENT BROKER, L. J. ROBSON, 32 Bishopsford Road, Morden, Surrey. Mitcham 2666.

EXCHANGE.

EXCHANGE : 3 tons ¾-in. "Isteg", 4 tons 1¼-in. and 6 tons 1½-in. round mild steel bars for equivalent quantities of 1-in., ⅞-in., or ¾-in. round mild steel bars. Apply RIBAR, LTD., Higher Swan Lane, Bolton, Lancs.

M.A., in collaboration with Professor H. W. B. Skinner, F.R.S. The main contractors were Bovis, Ltd.

The building is on two floors, one at road level and one 8 ft. below, and at 15 ft. below road level there is a pit for erecting the magnet. On the upper floor are an auxiliary equipment room, a room for a motor to lift a door weighing 30 tons, and a room for the ventilation plant.

The site was excavated to a depth of 30 ft. The pit was tanked with asphalt, the bottom was concreted to a depth of 2 ft., and plain concrete walls 6 ft. thick were built on two sides of this floor. As a further protection against radiation, pulverised sandstone is placed against the 6-ft. wall to a thickness of 10 ft. and is retained by a brick wall. The wall on the side adjoining an ante-room is 12 ft. thick, and the fourth side is built against rock. The floor at road level is 5 ft. 6 in. thick.

While experimental work is in progress the entrance to the cyclotron room (*Fig.* 2) will be sealed with concrete blocks. In these blocks the aggregate is steel punchings which produce concrete with a density of 300 lb. per cubic foot. Each block (*Fig.* 3) weighs 1½ tons, and they are laid to form a wall 6 ft. thick; 6 ft. of this class of concrete gives as much protection as 12 ft. of ballast concrete. About 1 ton of steel is used in each block. The blocks are made in a mould in which the steel-angled frames which form the arrises are first placed.

Fig. 3.—Blocks for Sealing the Entrance.

A Dam in Australia.

THE construction of a slab-and-buttress dam at Oberon, New South Wales, is described by Mr. H. S. Cornish, of the New South Wales Department of Public Works, in "Commonwealth Engineer" for November, 1951, from which the following is abstracted. The dam is a reinforced concrete structure 110 ft. high and 1300 ft. long at the top. The buttresses are roughly triangular in shape and are spaced at 18-ft. centres. The buttresses are 3 ft. wide at the bottom and taper to 1 ft. 6 in. at the top. The slab tapers from 4 ft. 3 in. at the bottom to 1 ft. 2 in. at the top. The buttresses are braced by horizontal beams.

The upstream faces of the buttresses slope at an angle of 45 deg., and the slab is separated from the buttresses by a layer of hard asphalt on the edges of the buttresses and a layer of soft asphalt on the middle portion.

Water Sealing.

The structure is founded on volcanic tuff having numerous heads and seams, and has a cut-off wall at the up-stream face taken a minimum of 10 ft. into solid rock. As trial bores revealed that leakage paths existed below the level of the cut-off trench, grouting was resorted to. Holes 10 ft. deep were drilled in the bottom of the cut-off trench at 3-ft. intervals on the centre-line of the trench. Intermediate 15-ft. holes were then bored and extended by pipes through the cut-off wall. After concreting the cut-off wall, the 15-ft. holes were grouted with 30 per cent. grout at 45 lb. pressure per square inch by a double-cylinder air compressor. Some of the 15-ft. holes took 160 cu. ft. of grout, but most required enough to fill the holes only, as did the 10-ft. holes.

Vertical construction joints in the cut-

off wall were sealed with 10-in. by 16-gauge copper strips cast 5 in. into each side of the joint. Above ground level, all horizontal joints in the face slab were similarly sealed, and vertical joints were sealed with 15-in. by 18-gauge crimped copper strips. Where vertical and horizontal copper strips in the face slabs crossed one another, the joints were welded, using copper strip as a filler; no flux was used to avoid electrolytic action. A further seal was provided at the vertical joints by leaving recesses 19½ in. by 6¾ in., symmetrically placed across them. These recesses were covered with ¾ in. of soft asphalt applied hot, and had 1¼ in. diameter brass bolts cast in them. Pipe ferrules were placed over the exposed portions of the bolts and 18-in. by 6-in. cover-slabs were cast in situ in the recesses.

Grain Silo for Turkey.

THE Direction Regionale de l'Office des Produits du Sol is inviting tenders for the construction of a grain silo of 10,000 tons capacity. The date for the receipt of tenders is March 31. Further information may be had from the Commercial Relations and Export Department, Board of Trade, Horse Guards Avenue, London, S.W.1 (quoting reference CRE/4086/52, TEN/3585/IBI).

Change of Address.

Messrs. Brown, O'Sullivan & Partners, consulting engineers, have removed to 123, Victoria Street, London, S.W.1. (Telephone, Tate Gallery 0266. Telegrams: Bronsul, Sowest, London.)

ESTIMATING PRECAST CONCRETE

A COMPLETE practical treatise on estimating and costing precast concrete, entitled "Estimating and Costing Precast Concrete Products and Cast Stone," by Mr. F. H. Fielder, Manager of Girling's Ferro-Concrete Co., Ltd. (Feltham), describes simplified methods based on the author's practice and long experience. Gives in handy form all the data necessary for estimating the cost of every type of precast concrete and cast stone unit, including the cost of materials, labour, overheads, and all other costs. 140 pages, 19 "at a glance" tables of costs, 54 worked examples, 49 illustrations.

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Rapid-hardening Portland, 8s. above ordinary Portland.

Aquacrete and 417, 32s. 6d. above ordinary Portland; paper bags included.

Colorcrete (buff, red, and khaki), in 6-ton loads, £14 1s. 3d.; paper bags included.

Snowcrete, £12 17s. 3d., inc. paper bags.

"Super-Cement," 32s. 6d. per ton above ordinary Portland cement; paper bags included.

High-alumina cement, 1 ton and upwards, 255s. per ton; paper bags 22s. per ton extra.

Snowcem paint, 71s. per cwt. inc. containers.

SHUTTERING.—For prices of timber, refer to S.R. & O., 1949, No. 1079 (price 1s. 1d.) and No. 94 (price 5d.) issued by H.M. Stationery Office.

REINFORCEMENT.—Mild steel bars, B.S. 785 (per cwt.); ½ in. to 2½ in., 35s. ⅜ in. to ½ in., 36s. 6d. ⅝ in., 37s. ¼ in., 38s. 6d.

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Columns, average 18 in. by 18 in. (per sq. ft.), 2s. 10d.; in narrow widths, 3s. 7d.

Beam sides and soffits, average 9 in. by 12 in. (per sq. ft.), 2s. 9d.; in narrow widths, 3s. 3d.

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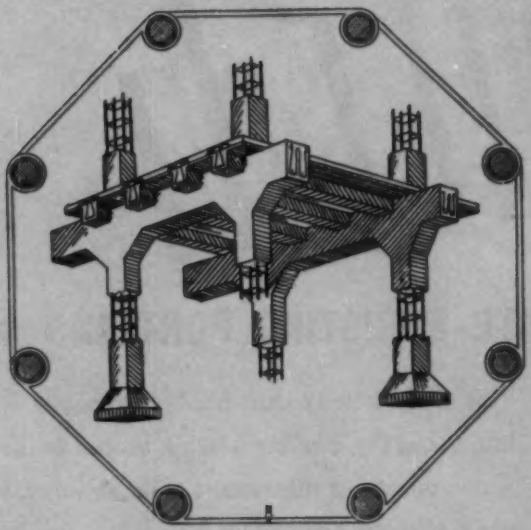

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